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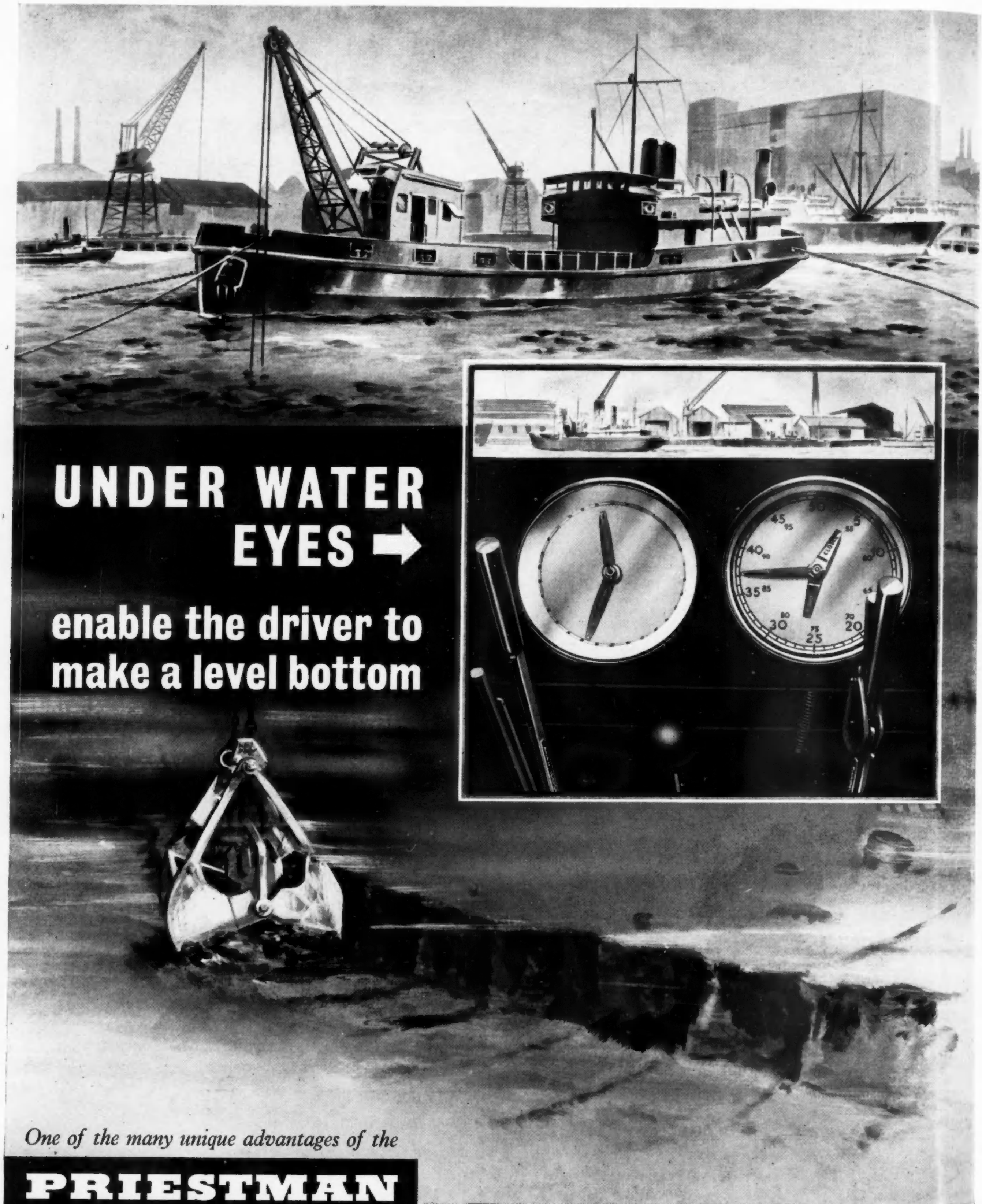
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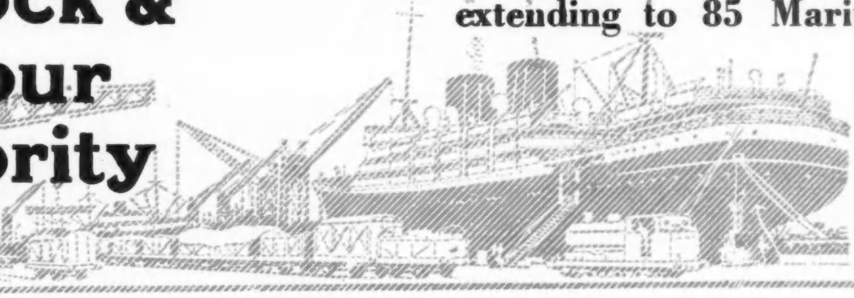


No. 455

Vol. XXXIX

SEPTEMBER, 1958

Monthly 2s. 6d.

Editorial Notes

Construction of Moturoa Wharf, Taranaki

The highly informative article by Messrs. W. S. Morrison and S. W. Butcher which we are publishing in this issue, gives details of the construction of a new heavy duty wharf for the port of Taranaki. This port lies on the west coast of the North Island of New Zealand about 160 miles north of the City of Wellington and is exposed to a maximum fetch (2,000 miles), well over the virtual limit (900 miles). Like many other ports of New Zealand, it is assailed by violent winds which interfere considerably with port operations. For this reason a windbreak 29-ft. high is constructed along the centre line of the wharf.

The main use of the new Moturoa wharf is to handle phosphates in bulk for which the design live load was fixed at the unusually high figure of 7½ hundredweight per square foot.

The clear exposition of the authors without unnecessary elaboration of common dimension features makes interesting reading and constitutes a sound report of current harbour engineering practice in New Zealand.

The most important factors governing the design of the wharf were (a) sturdiness and flexibility; (b) approach impact and repeated collisions due to ships working at their moorings during the ranging of harbour waters; (c) adequate fendering to prevent damage to ship or jetty and (d) more than the usual number of moorings ballards to hold the vessel securely to her berth and prevent lateral yawing

Hitherto very little has been done to evaluate the phenomena of harbour ranging in engineering terms, in spite of several investigations carried out on the full scale. Notable among the investigators was the late Professor Robert Knapp of the California Institute of Technology, who invented a useful apparatus for measuring the forces and ship motion during ranging.

Whilst wave action is undoubtedly the most important factor in the production of a ship's motion or working at her moorings the main direct cause is that due to the mass horizontal movement of the water. The horizontal component of the ranging wave is many times greater than its vertical component.

From observations made in the last decade the most damaging effects appear to happen within the lower range up to about 3 minutes period; with higher periods there is less chance of damage unless it coincides with the natural period of the harbour itself.

It seems almost certain that the harbour-master at Taranaki deals with a vessel's motion at her berth as a mooring job from the seaman's angle, an inheritance from those days when ships first commenced to berth alongside instead of *à pic*, in other words, normal to the wharf. The value of a good ship's mate is nowhere more useful than in this mooring ability which, when well done, renders the disagreeable ranging motion almost unnoticeable, even to dockers busy on board during the occurrence.

The decision of the panel who considered these various factors against the local conditions and the economic need was indubitably right and thenceforth the scheme was tackled boldly.

One feature of maritime and heavy engineering frequently forgotten, or unappreciated, in connection with New Zealand is the short supply and the high cost of skilled labour. It is a young country and the industrial resources have not yet had time to furnish the trained man-power and the technical cadre for its needs. Unlike the Crown Colonies, New Zealand has not had the benefit of years of Crown development. The prosperous state of the country is solely due to the foresight and energy of the New Zealander, and the Moturoa Wharf represents an example of construction comparable with the most modern of similar types in Europe.

Standardisation of Containers and Pallets

During recent years United States shipping companies have been steadily increasing their use of containers and, following research studies which gave evidence of the economies achieved by this type of cargo transport, the Department of Defence has purchased a large number of containers.

The need for standardisation of the sizes of containers is now widely recognised and a committee has been established to study the problem under the sponsorship of the American Standards Association, the American Society of Mechanical Engineers and the American Materials Handling Society. A report is to be made to the Federal Maritime Board which has endorsed in principle standardisation of container sizes for U.S. merchant marine use.

On a following page in this issue will be found a report of the fifth meeting of the I.S.O. Technical Committee, which was held in England recently to study further the implications of international standardisation of pallets.

Centenary Year of the Port of Copenhagen Authority

On the 30th December this year the Port of Copenhagen Authority will celebrate the centenary of the Act which established a Harbour Board for the Port in 1858. The foundation of the harbour itself can of course be traced back in history to the twelfth century at least, but this Act marked a special stage in the development of the Port introducing, as it did, a special port administration with the City Prefect of Copenhagen as Chairman and eight other members. A harbour master was appointed as head of the administration.

In 1913 this original Act was replaced by a new one which established the Port as a self-supporting, autonomous institution. The City Prefect remained as Chairman and the members of the Board represent the main interests attached to the port such as Parliament, the Ministries, the City Corporations, the shipping industry and commerce. As before, rates and regulations must be sanctioned by the Ministry of Public Works, who also have to approve any sale or mortgage of the Port estate.

In 1858 the total length of quays was about 13½ kilometres, to-day there are about 40 kilometres. The goods turnover has risen from 300,000 tons in 1858 to 8.5 million tons in 1957. Before the end of the nineteenth century, the most important single extension was carried through with the establishment of the Free Port, which was primarily built to secure the position of Copenhagen in the trade and traffic of the Baltic. It has been twice extended during the present century and is still improving and enlarging its facilities. In 1935 the oil harbour on the east coast

Editorial Notes—continued

of Amager was constructed and to-day covers an area of 100 acres.

In view of a continued expansion in trade, the port is now concentrating its activities on installing more and better mechanical handling equipment to achieve faster loading and unloading. A problem the Harbour Board is currently facing is the conflict between land traffic crossing the fairway and the sea traffic to the South Harbour. To relieve this, a third bridge between Sjaelland and Amager is in course of construction, but it is believed that the most rational—although most expensive—solution would be a deep channel through which ships could enter the South Harbour from the south, thus avoiding the two northern bridges. This proposal, however, is still very tentative.

We hope to publish at a later date an account of the development of the Port and of its present facilities. In passing, we would draw our readers' attention to the publication last month of the anniversary issue of the Copenhagen Harbour Journal. This journal, which has now been in existence for ten years, contains in this special issue five authoritative articles on the port, with a colour map and many interesting illustrations. English translations of the entire contents are included.

Study of Range Action at Cape Town

French engineers from the hydraulic research laboratories at Grenoble have arrived in Cape Town to collaborate with engineers of the South African Railways and Harbours Administration to carry out further experiments in connection with alleviating range action in Table Bay Harbour. According to recent press reports, the pilots at Cape Town have urged that the entrance to the Duncan Dock should be widened from 400-ft. to 600-ft. in order to provide more clearance for big ships and their accompanying tugs. The range action in the winter is severe, however, and it is feared that if the entrance were widened by so much, the ranging would be even worse.

It is expected that the French research engineer M. Jean Bard will stay for one or two months directing the siting of the experimental equipment. Mr. Willem C. Q. Joosting, district engineer of research, South African Railways and Mr. Alexander Wilken, district harbour engineer Cape Town, are collaborating with M. Bard. At Grenoble experts will study a model of Table Bay and the harbour from which they hope to work out a solution to the problems. Meanwhile, a similar model will be studied in Johannesburg by South African engineers, and it is hoped that plans will be put into operation within two years.

As a matter of interest, it should also be mentioned that the Hydraulics Research Station at Wallingford in Great Britain is at present making a special study of the problem of ship ranging, in relation to the difficulties experienced in mooring securely.

Limitation of Shipowners' Liability

Probably the most important single provision in the Merchant Shipping (Liability of Shipowners and Others) Bill which recently received the Royal Assent, is the raising of the monetary levels at which, provided they are not guilty of actual fault or privity, shipowners can limit their liability for claims for personal injury and damage to property. The present limits of £15 per ton (and £8 per ton if there are only property claims) were first laid down in 1862. They have long been recognised as out of date and have in fact given rise to hardship, particularly in accidents to small vessels. The Act raises the figures to £74 per ton and £24 per ton respectively and a special minimum tonnage provision ensures a reasonable compensation fund for death and personal injury even with the smallest vessel.

Other provisions of the Act extend the types of liability which can be limited and extend the categories of person entitled to limit their liability, notably to include the master and members of the crew and any person interested in or in possession of the ship. There are also provisions for the release of ships from arrest if security up to the limits of liability has already been given elsewhere and is actually available to the claimant.

With one exception, all the provisions of the Act will come into force immediately, and it can be extended later by Order in Council to British overseas territories. The Act—the latest of a long series of Merchant Shipping Acts stemming from the great consolidation measure of 1894—gives effect in the United Kingdom to the International Convention on the Limitation of Liability drawn up by the Brussels Diplomatic Conference in October, 1957. This Convention offers the first real possibility of general agreement in one of the most complex fields of maritime law. For the first time also it is based upon the British system of limiting liability according to the tonnage of the ship rather than according to her value.

To ensure that the changes in the law were acceptable to all the parties concerned and would be workable in practice, the United Kingdom Government carried out long and intensive consultations with the interests affected by the Bill, starting before the Brussels Conference. Amongst the bodies which gave their views were the shipowners' organisations, trade unions, shipbuilders, insurance, legal and commercial interests, and harbour authorities.

The Convention has been signed by seventeen countries including most of the great maritime powers. The United Kingdom is now in a position to ratify it and it is hoped that other countries will follow the lead that has been given. The Convention will come into force internationally as soon as ten States have ratified it including at least five with over a million tons each of shipping.

Centenary of the Discovery of Lakes Victoria and Tanganyika

The discovery of Lake Victoria, the second largest lake in the world, was made on August 3rd, 1858, by John H. Speke after he and Richard Burton had on February 13th of that year discovered Lake Tanganyika. To-day on these two lakes, as also on Lakes Albert and Kioga and the River Nile, the East African Railways and Harbours provide regular lake and river services for both passengers and cargo. Some 330,000 tons of cargo and 713,000 passengers were carried on these inland marine services during 1957.

A new fast passenger ship for service on Lake Victoria is currently under construction in Glasgow, where she should be completely assembled by May, 1959. To be named the R.M.S. "Victoria," she will be the biggest and fastest ship on the African lakes. Her speed will enable her to make two trips a week round Lake Victoria, each of some 700 miles. In addition to excellent passenger accommodation, the ship will provide refrigerated cargo space and will carry cars and mail. A new port is being built at Mwanza, facilities at Bukoba are being enlarged and, also, a 300-ton oil tanker is under construction at Kisumu. In all, more than 6,000 miles of steamer services are operated by the East African Railways and Harbours.

North Foreland V.H.F. Service

On Friday, September 5th, a new ship-to-shore radio telephone link came into service through the G.P.O.'s coast radio station at North Foreland. The service was opened when the Rt. Hon. Ernest Marples, M.P., Postmaster General, received the first call from Viscount Simon, Chairman of the Port of London Authority, who was on the Marconi Company's yacht "Elettra II" cruising in the Thames estuary.

The new service will enable suitably equipped ships within about 40 miles of North Foreland to contact any telephone subscriber in the United Kingdom. It will be particularly useful for ships entering or leaving the Thames or passing between the North Sea and the Straits of Dover. The service will be open to both British and foreign ships, and can be used by passengers and crews on ships providing the necessary facilities. The shore equipment complies with internationally recommended technical standards and two channels will be used with frequency modulation. Charges for a three minute call will be 6s. 6d. for combined coast station and ship charge, plus a landline charge of 6d. or 2s. 6d. depending on the distance between North Foreland and the telephone subscriber called. Further V.H.F. services are planned for 1959 at Niton (Isle of Wight), Humber and Lands End.

Modern Dry Docks

The series of articles on "Modern Dry Docks; their Design, Construction and Equipment" has been unavoidably postponed for this month, but will be continued in future issues.

The Port of Taranaki, New Zealand

Design and Construction of Moturoa Wharf*

By W. G. MORRISON, O.B.E., E.D., B.E., M.I.C.E., M.N.Z.I.E., and G. W. BUTCHER, M.C., B.E., STUD.N.Z.I.E.

History

PORT Taranaki is a breakwater harbour and New Zealand's fifth port in order of tonnage of goods handled. The breakwater was commenced in 1879 and for a short time was the only berthage available, there being no wharves at that time. The original Moturoa Wharf was erected in timber (mainly totara) eight years later as a cattle wharf. Sir John Coode who was associated with a large number of New Zealand harbours, especially the very difficult ones on the west coasts of both islands, was consulting engineer to the Board for these early works.

The original wharf was extended during 1890 to 1900 and futher extended and widened in 1909-10 and in 1913. The variations in timber used in these extensions and differing types of construction were very noticeable during the demolition to make way for the new wharf.

A second wharf (Newton King) was built in Australian hardwood during 1925-26.

Plans were prepared about 1929 for the replacement of the Moturoa Wharf with a reinforced concrete structure but for various reasons the work did not proceed. Even at this time some concern was felt about the condition of the wharf. By 1953, however, the wharf had reached such a state that rail traffic could no longer use it.

Tenders for the replacement were finally called in September, 1954.

The new wharf has a width of 90-ft. and a face length of 1,100-ft. 6-in. on the east side. The berthage provided is as follows:

East Side: (1) An overseas berth which is intended for bulk cargoes, mainly phosphate. This berth will be equipped with 3/5 ton electric cranes and will eventually be dredged to 36-ft. L.W.O.S.T. There are four rail tracks alongside the berth.

(2) A dredge berth dredged to 26-ft. L.W.O.S.T.

West Side: An inter-colonial berth dredged to 26-ft. which can accommodate two coasters. Two rail tracks are provided.

Geology of the Site

The harbour area, and indeed a considerable portion of north Taranaki, consists of what can best be called volcanic debris. This material is believed to have originated from a volcano or group of volcanoes situated in which is known as the Pouakai Ranges to the south-west of New Plymouth. This mass of volcanic debris, together with the somewhat earlier lava flow extrusions forming the prominent Sugar Loaves in the port area, are termed Pouakai Series. Bulletin No. 14 (New Series) of the N.Z. Geological Survey describes the series as consisting of a succession of beds generally of fragmental volcanic material, sometimes water-worn but usually angular or subangular, with which are interbedded smaller layers of thoroughly water-worn pebbles of quartz, greywacke and argillite.

Agglomerates and volcanic breccias containing volcanic boulders up to 10-ft. in diameter are exposed in the sea cliffs in the area, together with fine-grained materials, claystone masses of considerable size, and sandstones.

From the water-worn character of the majority of the boulders and the bedding exhibited, it is believed that the series were accumulated under water. The haphazard nature of the materials of the harbour bottom are shown in the test bores which encountered hard andesitic rock, a softer rock, sandstones, mudstones, papa (claystone) conglomerates, boulders, gravels and sands.

Part I: Design (W. G. MORRISON)

1. General Planning

THE position, alignment, width and facilities to be provided were laid down by the Taranaki Harbour Board. Some members of the Board were very interested in the possibility of having a solid wharf. (It would be more correct to describe the work as a jetty but local usage has established the term "wharf"). The conception of a solid wharf appealed as one which would carry unlimited deck loads and which would require a minimum of maintenance. This led to an interesting comparison of types. The first was that of two parallel gravity walls, but since these would have to be about 60-ft. high and perhaps 25 to 30-ft. thick at the base, the cost would have been prohibitive. The next type investigated was that of parallel walls of sheet piles in steel or concrete, but no standard steel sheet pile would have been strong enough, and it was calculated that 24-in. by 7½-in. steel joists on edge and practically touching would be needed. Large concrete caissons filled with sand were also considered.

It became apparent early, however, that the cost of a solid wharf would be two or three times that of a piled wharf and in addition the construction difficulties would have been enormous. An estimate was, therefore, taken out for a piled wharf, the choice being between an all-concrete wharf, an all-timber wharf or of a wharf with timber piles and concrete deck such as is finding favour in some Australian ports. The Moturoa Wharf is near the New Plymouth breakwater, over which green seas break in bad weather, and there is considerable range at the berths. The conditions thus call for a construction which is particularly stable against the impact of ships and for this reason the scheme of timber piles with a concrete deck was rejected. This narrowed the field down to an all-timber or an all-concrete piled type.

In view of the requirement that the wharf would have to withstand unusually heavy impact from ships without damage to the wharf or to the ship, some local opinion was strongly of the opinion that a timber wharf would be more suitable in that it would be more flexible. This would hold good if the timber wharf were braced by means of diagonal bracings and walings to about low water level, but such construction is notoriously difficult to maintain, particularly under attack from *Teredo* and *Limnoria* such as occurs in Port Taranaki, and the modern trend is all towards the use of timber raking piles to give stability.

It is doubtful if a timber wharf with raker piles is appreciably more flexible than a similar concrete wharf, and there are difficulties in making the connections at the junction of the rakers with the deck strong enough for the rakers to function as designed. In any event it would have to have a special fender system of much the same type as would a concrete wharf.

Partly on these grounds and partly because of anticipated difficulties in getting deliveries of the quantity of long timber piles required, it was decided to have an all-concrete wharf. Maintenance was also considered, but, contrary to popular belief, the evidence was that the cost of maintenance of a concrete wharf is not much less than that of a timber wharf. The cost of construction was estimated to be about the same.

At the stage when designs were completed, the Board, wishing to allay a certain amount of criticism by some local interests, set up a panel of engineers to review the decisions reached and also to report on future planning of other port facilities. This panel consisted of D. S. G. Marchbanks, Engineer to the Wellington Harbour Board, C. R. Davis, Engineer to the Napier Harbour Board, and the writer. The panel endorsed the scheme which had

* Paper presented to the New Zealand Institution of Engineers. Reproduced by kind permission.

Port of Taranaki—continued

been prepared for the wharf, and in particular pointed out that, if a fire were to occur which would put the wharf out of commission, the effect on the trade of the port would be most damaging as there is, and will be for years, only one other jetty (the Newton King Wharf).

The panel also endorsed the fender system in principle so that the way was clear for design to be completed and for contract documents to be drawn up.

2. Design

2.1. Deck

The live load specified was 840 lb./sq. ft. or Ka locomotives on tracks at 11-ft. centres. Crane tracks were to be provided for crane spans of 35-ft. 9-in. along each side with a central windbreak. The total width of deck was to be 87-ft. exclusive of fendering. In the early stages it was planned to have a central trough in the deck for a conveyor belt to handle the bulk phosphate which forms a substantial proportion of the cargo discharged at the wharf. This conveyor trough was not adopted in the final construction. Further, the rail layout was altered from time to time.

The first consideration, that of spans, was decided largely by pile sizes. The longest piles would be 80-ft. long and if larger than 18-in. x 18-in. would require very special gear for handling and driving. The load capacity of 18-in. by 18-in. piles was judged to be such that piles would have to be at about 12-ft. centres both ways. On the basis of 12-ft. spans, various arrangements of beam and slab were designed and estimated with a preference for a minimum of beams below slab level to reduce corrosion and maintenance. Beams were used to carry the crane rails and advantage was taken of these beams to provide resistance points for the tops of the raker piles.

During the progress of construction, it was decided to omit the crane on the western side and to increase the span of the crane on the eastern side so that only two beams out of the four in the wharf would carry crane loads. The windbreak was correspondingly moved to the position shown on the drawings. The deck was also designed to act as a horizontal plate girder in distributing the impact of ships to a sufficient number of bays, but this aspect is considered more fully under section 3, "Fenders."

2.2. Vertical Piles

A preliminary comparison was made of the cost of square piles 18-in., 20-in. and 22-in. assuming that the load capacities would be 70, 100 and 128 tons respectively. These loads were calculated for the piles as long columns. Reinforcing steel was based on lifting the piles at two points or for $M = Wl^2/48$. This study showed the cost per ton carried to be very nearly the same for each pile size, and 18-in. was selected as the pile size because larger piles would require heavier lifting and driving gear. Even so the weight of an 80-ft. pile of 18-in. size was 12.5 tons.

The piles were reinforced with eight 1-in. round rods for the full length. Saurin (Jour. I.C.E., March, 1949) considers this is light for English practice and recommends $M = Wl^2/18$. The percentage of longitudinal reinforcement for eight 1-in. rods is 1.94 but since, in each plane of bending, only six rods are effective, it is better stated as 1.455. Building Research Technical Paper No. 20 states: "It was found by experiment that the proportion of main steel did not seem to have any effect on the resistance of piles to driving stresses." American practice seems to be to use less steel but to take more care to avoid stresses in handling. During construction there was no evidence that the piles did not have enough steel for handling, but their "whippiness" during driving caused some concern and the reasons are discussed in the next paragraph.

A fundamental point about the design of piles is that, if they are designed for a factor of safety of, say, 3 against static loads, and if they are driven to a calculated resistance of three times their static load, then the factor of safety during driving must be about 1. For short piles, the concrete quality can easily provide a factor of safety of more than 3 against static loads, but for piles which have a long unsupported length the margin must be fine unless the piles are made so big as to add greatly to the cost.

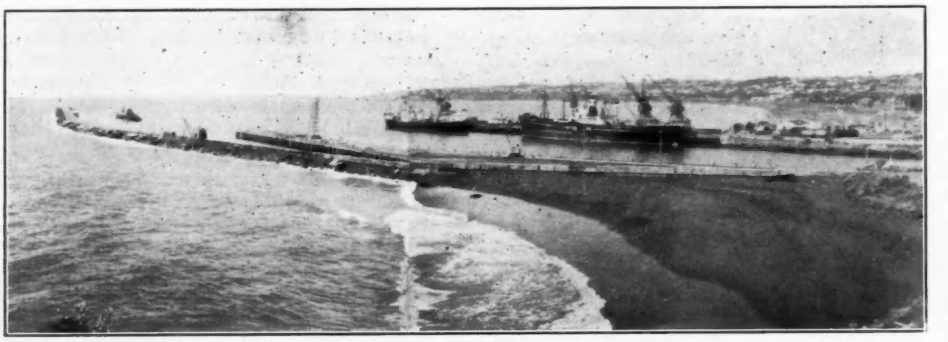
The final calculated load for interior vertical piles was taken as for a live load on the slab of 6 cwt./sq. ft. though the slab itself was designed for a greater load on account of uneven distribution. Adding the weight of an 80-ft. pile at $12\frac{1}{2}$ tons makes the load at the pile point approximately 70 tons, and if for calculations of pile buckling only half of its own weight is taken, the long column load is 63 tons. According to N.Z.S.S.95, this is safe on a pin-ended length of 51-ft.

In practice the piles did not drive as far as was expected and the deepest was to R.L.—10 in bent No. 3 row F. This pile in position is in water approximately 42-ft. then in soft material for another 8-ft. and in harder material for another 20-ft. It is thus 53-ft. between ends which have a fair degree of fixity and should therefore be safe enough. As a check, the Euler crippling length between pin ends for 63 tons for $E = 3{,}000{,}000$ lb./sq. in. is 61-ft. and for $E = 4{,}000{,}000$ lb./sq. in. the crippling length is 71-ft. If such a pile is driven to a factor of safety of 3 the instantaneous load becomes 210 tons. For such a load the pile is barely adequate as a short column so that the following courses would be open: (a) Make the piles strong enough. This would need 27-in. piles and nearly treble the cost. (b) Reduce the factor of safety in driving. To some extent this was done. (c) Assume that the driving impact would be of such short duration that the pile would not have time to buckle. In fact the longest piles did vibrate to an undesirable extent if driving resistance exceeded a certain amount, and considerable judgment had to be exercised to prevent the piles cracking. Some did.

A static loading test was envisaged before the contract was let, and reviewed several times during construction. No such test was made, partly because the bottom was so variable that a single test would not have given any general indication and partly because all the piles, as far as the records showed, would have had to be driven only a few inches further to take a much firmer driving set.

2.3. Raker Piles

Various arrangements were considered on the basis of four rakers per bent, and the layout adopted was the best that could be devised to distribute the load of both vertical and raker piles as uniformly over the width of the bent as was possible. The heads of the rakers were made to come under the crane beams, so that the crane beams would serve as strong points in distributing horizontal loads to the deck system and so that the rakers would assist in taking the vertical loads from the cranes. On the basis of assumed horizontal loads as discussed under "Fenders," the maximum and minimum loads per pile were calculated and plotted on Fig. 1. This diagram shows a maximum load of 62.5 tons on vertical piles in row D and a maximum load on the rakers of 77.7 tons in row A. The design loads will never be realised since the assumptions involve crane loads in an



General view of Port Taranaki from Mikotahi, showing the position of Moturoa Wharf in relation to Newton King Wharf. Construction of the new wharf shown in its early stages.

Port of Taranaki—continued

almost impossible position which is not in the least likely to take place under conditions of maximum ship impact.

The test bores showed the bottom to be so variable that it was a difficult matter to know how many piles to cast of each length and it would have greatly facilitated operations if rakers could have been of the same type and interchangeable with the vertical piles. The contract was let on this basis, but as soon as the first long vertical piles were driven the vibration was deemed to be too much to chance driving rakers of the same design, and a change was made for rakers to be 24-in. by 16-in. similar to those used on the Auckland Import Wharf. The longest rakers made were 70-ft. with five 1-in. rods on each 16-in. face and two 1-in. rods on each 24-in. face making 14 rods in all. For shorter piles the steel was reduced. The moment of inertia of the new design is more than twice that of the 18-in. square pile but even so it will not stand up to calculated bending stress plus driving resistance of 220 tons for a long column. Nevertheless, these rakers were in practice quite satisfactory and did not vibrate during driving so as to cause concern.

After the event it is easy to speculate that with such long piles it would perhaps have been better to use heavy steel joists encased after driving in concrete from mud to deck level.

2.4. Windbreak

Previous windbreaks used at Port Taranaki had been of triangular timber frames. For the Moturoa Wharf an open portal in steel was used so as to create space for wharf offices and small stores. The wind pressure was taken as 30 lb./sq. ft. and the calculated deflection at the top is 2¾-in. The bottom 20-ft. is clad in G.P.M. and the top 5-ft. is of horizontal timber planking.

2.5. Bollards

These are of cast steel designed to take a pull of 75 tons at an upward angle. This is calculated as the breaking load of two of the coir springs commonly used at the port.

3. Fenders

The previous sections of this paper have been very much condensed in order to make space available for a more detailed study of the fender system. The idea that a concrete wharf is inherently too rigid for the conditions at the port has already been mentioned. Theoretical considerations dictate that, in order to absorb the momentum of heavy objects without undue force, the resisting forces must act over a distance.

3.1. Selection of Fendering

On the assumption that the Newton King timber wharf had proved satisfactory, an analysis was first made to see what it could take by way of energy absorption. It had been stated that at times the wharf deflected laterally as much as 3-ft. 6-in.

This wharf consists of pile bents at 12-ft.

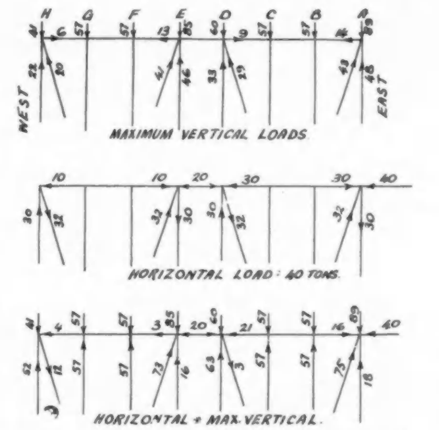


Fig. 1. Pile loads (tons). Note horizontal plus minimum vertical gives uplift "A" 22 tons.

centres, with 14-in. piles at 8-ft. 6-in. centres in the bents. There are nine such piles, and at each side of the wharf is an extra fender pile integrally connected with the wharf structure, making the wharf approximately 75-ft. wide overall (see Fig. 2). Transverse bracing consists of a double 10-in. by 8-in. totara waling 13-ft. below the deck, and diagonal bracing of single timbers both ways, and across two pile

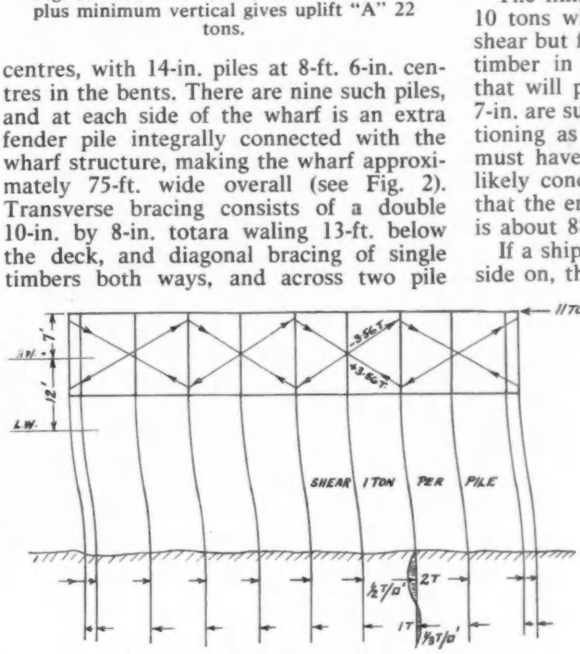


Fig. 2. Newton King Wharf : Case I.

spaces so that in the width of the wharf there are four sets of double diagonal braces. The bracing connections consist of single muntz metal bolts of (originally) 1⅛-in. diameter. The strength of the timber in the bracing is not critical except as regards the bearing of the bolts.

Preliminary calculations showed that, under assumed design conditions, the deflection of the wharf could be nothing approaching 3-ft. 6-in. so various extreme assumptions were made in order to assess the energy-absorbing properties of the wharf and to determine whether the deflection alleged to have been observed could be reconciled with calculations.

These assumptions were:

Case I: Bracing presumed effective. Piles assumed "fixed" at a level 5-ft. below bottom or 30-ft. below bottom walings.

Case II: Bolts in diagonal bracing assumed 1-in. slack. Piles assumed to rotate 1 in 100 at a point 4-ft. from the toe, thus giving a length of pile in double flexure of 36-ft.

Case III: Bracing assumed completely ineffective—i.e., bolts sheared or very slack. Piles assumed to act as simple cantilevers on a length of 50-ft.

Case IV: Bracing effective but piles slack in the ground and acting as cantilevers 41-ft. long below the bottom waling.

Calculations for one pile bent may be summarised as follows:

	Load (tons)	Deflection (in.)	Energy (in. tons)	Notes
Case I	11	2.3	12.7	Safe load
Case I	33	7	115	Limit of bolts
Case II	—	9	—	Pile slack
Case II	28.4	15.2	88	Limit of bolts
Case III	11	35	192	f_b on piles 4,900 lb.
Case IV	14	25	175	Limit of bolts

The limiting bolt load above is taken as 10 tons which is possible for the bolts in shear but far too much for the surrounding timber in bearing. The only assumptions that will permit a deflection of more than 7-in. are such that the structure is not functioning as designed and such that racking must have a bad effect. Perhaps the most likely condition is Case II which suggests that the energy which one bent can absorb is about 88 in. tons as an outside figure.

If a ship moves against this wharf broadside on, then perhaps 30 pile bents will be involved, with the ability to absorb 2,640 in. tons, but a more likely condition is for the ship to contact towards one end, in which case fewer bents will be effective and forces must be transmitted longitudinally per medium of the deck bracing.

However, the deductions which can be drawn from an analysis of the Newton King Wharf are not very helpful except to show that a structure of very limited capacity has been deemed adequate to absorb the forces imposed by ships in berthing or while tied up alongside.

As a general approach to the problem of fendering, Little (Proc. I.C.E., Feb., 1953) states: "It is almost certain that no matter how well a fender has been designed and however well it may stand up to every-day usage, it will one day receive a blow that will destroy it. And the range between normal usage and a not unreasonable accidental possibility is so high that it is not really practical to cater for the latter." Minikin, in *Winds, Waves and Maritime Structures*, presents the result of a questionnaire on the subject of approach velocities and states that on 49 records the velocity normal to the wharf for the last 10-ft. of travel exceeds 1 ft./sec. in only one case. Three-quarters of the cases recorded give a velocity of 2½-in. to 4-in.

While the case of vessels out of control approaching bow on or nearly so cannot be disregarded in choosing the best type of fender system, it cannot in practice be catered for without accepting local dam-

Port of Taranaki—*continued*

age. At the Moturoa Wharf, the alignment and layout of the turning basin make this a remote possibility and in practice it is found that the normal hazard is not that of berthing ships, but of holding them alongside when tidal and weather conditions cause heavy ranging. This emphasises another well-known fact: many fender systems are quite adequate to resist forces normal to their face, but are subject to frequent damage from forces along the face—i.e., due to the moored ship working fore and aft.

The Moturoa Wharf is expected to berth overseas vessels on the eastern side only, so all subsequent observations will relate to the eastern berth. (The fender system on the western side is a scaled-down replica of that on the east.) Minikin at least gives a starting point in respect of forces, and this was supplemented by observations of a ship "working" at the berth under adverse conditions. These showed the ship to come against the wharf at an angle of 1 in 50 at what was estimated to be 3 ft./sec. For a 12,000 ton ship at this velocity, the energy of movement is 1,680 ft. tons. Theoretically the energy transmitted by an end blow is one quarter of the above. The angle of 1 in 50 means that the blow must be near the end so it is reasonable to reduce to 500 ft. tons. Minikin quotes tests to show that, for a stiff braced jetty, the energy absorbed by the jetty is about half that of the moving ship so the figure is again reduced to 250 ft. tons or 3,000 in. tons. If some such figure must be catered for on perhaps 10 bays, the fender system per bay should be able to absorb 300 in. tons. For such a condition, vertical concrete piles are useless. For stress in reinforcing steel of 12,000 lb./sq. in. the static load is calculated as 4 tons, the deflection 0.9-in. and the energy 1.8 in. tons. With four rakers static load is 100 tons, deflection of the order of $\frac{1}{4}$-in. and energy 12 in. tons.

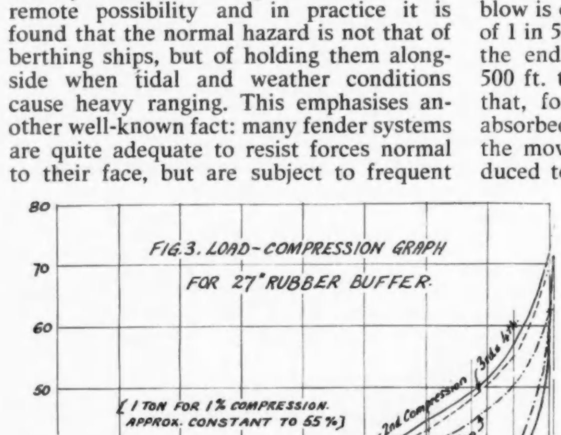
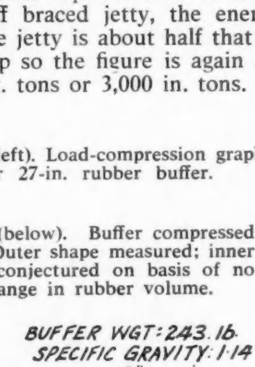


Fig. 3 (left). Load-compression graph for 27-in. rubber buffer.

Fig. 4 (below). Buffer compressed 60%. Outer shape measured; inner shape conjectured on basis of no change in rubber volume.



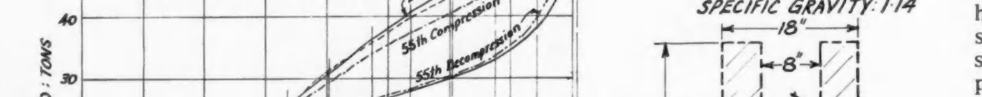


Fig. 5. Typical impact of ship working at moorings.

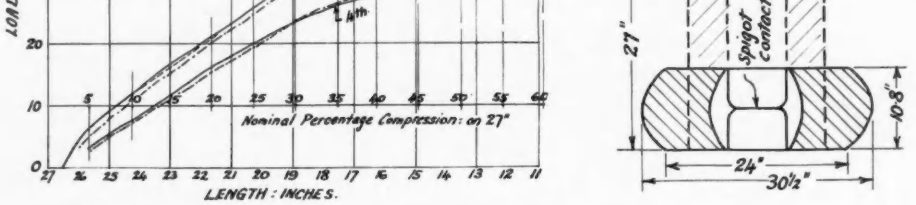


Fig. 6. Bending moment in wharf owing to impact of ship.

Obviously a wharf on concrete piles must have a special fender system. A timber piled wharf was calculated to take per bent a static load of 72 tons, with deflection of 12-in. and energy of 432 in. tons. This, however, assumed bending stresses in the piles of 3 tons/sq. in. which is very high even for Australian hardwood, and it required bracing to low water level of a type which has proved difficult or impossible to maintain, and which would require connections vastly better than those commonly used.

Gravity fenders were considered, and in principle a system can easily be devised—e.g., a weight of 25 tons lifted 18-in. provides energy absorption of 450 in. tons. However, the cost was estimated to be very high and the mechanical problem of ensuring that gravity fenders would function properly in resisting longitudinal forces was deemed to rule out this type. Torsion bars were analysed but found to be incapable of taking the forces unless of fantastic dimensions. Little points out the capacity of timber as an energy absorber and it has in bending great capacity, but once the decision was made to have a concrete wharf it did not seem possible to incorporate timber in the fender system so as to make much use of it for purely energy-absorbing purposes. It is, of course, incorporated in the design quite extensively for practical reasons.

With a wharf on concrete piles and with rakers, the only place to take impact is at deck level and the choice narrowed down to an arrangement of timber fender piles with stout longitudinal walings as a practical system, with some special energy-absorbing devices between the timber and the edge of the concrete deck. The contract was first drawn up so as to use helical steel springs at 6-ft. centres, each to take a force of 50 tons for a travel of 3-in. and thus absorb 75 in. tons. Admittedly, this is not enough in the light of earlier calculations but it was thought to be as good as the Newton King Wharf and the cost would have been about £10 per foot of wharf face which was thought to be high enough.

The writer's connection with the work commenced in December, 1950, and all the original design was done in the following year. Tenders were not called for until the latter half of 1954 and, though the helical steel springs were specified, the writer had meantime read Mr. Little's paper and also received other information which suggested that it would be advantageous to consider the use of rubber as an energy-absorbing medium. In everyday use it is, of course, the energy-absorbing material par excellence.

3.2. Rubber for Buffers

An investigation of the possibilities of rubber showed that it would meet requirements in respect of impact and facilitate logical investigation of the effect on deck, piles and the structure generally, without the cost being any higher than for steel springs. Advice was then sought from various authorities and manufacturers as to the quality and shape which would be most suitable for the buffers, and it was found that there was quite a divergence of views. Rubber is frequently used for buffers in shear, generally bonded to steel, but appear to be expensive per unit of energy

Port of Taranaki—continued

and seem to have so many steel parts that they must be difficult to maintain.

Rubber in simple compression is the most obvious application and was adopted. Then it was found that some favour a practically solid cylinder for the reason that a hollow cylinder is likely to buckle or form cusps in the rubber when it is compressed. From the design point of view, there may be advantages in a solid cylinder, but the curing of the rubber must be uniform, and for the comparatively large mass required the range of firms able to cure the buffer is limited. Others stated that a rubber extrusion is the only proper method of manufacture, and that if the cylinder were to be mandrell wrapped there would be a separation of the piles under repeated compressions.

The supply issue was decided on price. The smaller buffers for the west side were made by an extrusion process by Goodyear Rubber Co. in England. The larger buffers were made in Auckland by Reid Rubber Co. whose offer was accepted partly on price but also because it enabled inspection and tests to be made. The process was to wrap a thin sheet of rubber on to a heated cylinder and so build up to the required dimensions.

The buffers for the eastern side were hollow cylinders 27-in. long, 18-in. outside diameter and 8-in. inside diameter. They were required to stand compression of 60% of their length, with a load at that point of approximately 600 lb./sq. in. of original section. The load to compression ratio was required to be nearly a straight line. Provision was made for testing selected buffers with 50 cycles of compression of 60% (i.e., to 40%) of original length.

Before going into production, a trial buffer was compressed 175 times without any sign of cusping or damage of any kind. Later a buffer was compressed 350 times and seemed to be in perfect condition. All tests throughout manufacture were satisfactory, though one buffer did show slight peripheral folding just next to the bearing plates. The severity of the tests is to the layman quite remarkable since the rubber is to the touch similar to that used for rubber heels, not by any means to be compared with sponge rubber.

TABLE I: CHARACTERISTICS OF IMPACT SHOWN IN FIGS. 5 AND 6

Buffer No.	Compresson (in.)	Load (tons)	Energy (in. tons)
1	3	11	16
2	5	18	45
3	8	30	120
4	12	44	264
5	14	52	364
6	16	75	494
7	15	55	413
8	13	48	312
9	11.5	43	248
10	10	37	185
11	8.5	31	132
12	7	26	91
13	5.5	20	55
14	4	15	30
15	2.5	9	11
16	1	4	2
	Total	518	2,782

A graph showing load-compression characteristics given in Fig. 3 and the shape of the compressed buffer in Fig. 4. Without being able to ascertain the shape of the interior of the buffer at 60% compression, it is not possible to say to what extent the rubber is made to occupy a smaller volume and to what extent it is merely distorted. The indications are that the volume is not much affected. The volume of the original hollow cylinder is 5,500 cu. in. When compressed to a length of 10.8-in. (to 40%), if the inner diameter remains unchanged the mean exterior diameter should be 26.75-in. From measurements of the compressed shape it is 28.375-in. so that no reduction in volume is indicated. If the volume is unchanged, the interior shape is probably as shown in Fig. 4.

The method of mounting the buffers on spigots is worth note. It is designed to allow some longitudinal movement, while the spigot shape is of some significance in protecting the buffers from forming cusps during compression. The buffers bear directly on the timber of the walings at their outer ends and the inner ends bear on a galvanised steel plate 3-ft. in diameter to protect them from abrasion by the concrete. They are maintained in a state of slight initial compression by the chains from the concrete wharf to the fender piles.

3.3. Review of Calculations

Earlier reference is made to ship energy of 500 ft. tons. of which the wharf is required to absorb half. Just how this reduction may be calculated is not clear, but the ship itself is flexible as a whole, to an extent which would allow perhaps 3-in. deflection on impact. Also, the ship's plates locally will deform under excessive pressure. Another factor is the inertia of the wharf. The weight of, say, 350-ft. involved is upwards of 4,000 tons. At any rate the reduction will be assumed, so that the energy of impact to be absorbed by the wharf is 250 ft. tons or 3,000 in. tons.

The timber fender piles have a negligible effect. They were not driven with an outward rake as is sometimes done, but were driven vertical because they would in that way offer less resistance to pressure from the bilge keels of ships rolling at the berth, and be less likely to suffer damage. At a depth of 25-ft. below high water they can on reasonable assumptions be pushed sideways about 8-in. without being overstressed. The first resistance offered to the ship at Port Taranaki is from old tractor tyres fixed over the walings. Tests show that these absorb about 47 in. tons in being compressed 9-in. but this will be neglected in subsequent calculations. It is by no means negligible and the tyres also serve a useful purpose in restraining longitudinal movement by friction.

On the assumption that a typical 12,000 ton ship hits the wharf at an angle of 1 in 50, the effect of the walings must first be considered in distributing the impact among a number of buffers. It would not be practicable to make the walings strong enough to act as horizontal beams of appreciable length in view of the forces involved. On the contrary, there is a case for making them flexible enough to follow the curve of a typical ship without overstress. This was borne in mind in their design. They also have to be strong enough to act as beams on 12-ft. span so that, when the ship contacts the chafing pieces on the timber piles at 12-ft. centres, they can compress the intermediate buffers at 6-ft. centres. This is

Vessel coming alongside contacting fender walings amidship. The high windbreak is seen to the right.

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East side of wharf showing close spacing of bollards and timber planking spaced apart at top of windbreak.

why they are keyed together to resist longitudinal shear consequent on the bending action. The walings will, beyond the contact length of the ship, have to be strong enough to compress some buffers to a diminishings extent until they are clear of its influence.

Another function of the walings is in respect of longitudinal movement of the ship. They now function as ties to distribute the forces as far as possible along the wharf, and are therefore connected strongly together, their tension value being about 37 tons. It is of interest that buffers remote from the contact length of the ship play a part in resisting longitudinal movement, since, if the walings move longitudinally, the action of the diagonal chains is to compress the buffers to approximately the same extent as the waling moves.

If the buffers are made to conform to the shape of the ship as in Fig. 5, their compression and energy absorption are as shown in Table I, which accounts for approximately the 3,000 in tons assumed. Note that buffers 2 and 3 must be compressed by the beam strength of the walings rather than direct ship-to-waling pressure. If, now, the horizontal load to be taken by the piles of each bent is not to exceed 40 tons as previously assumed for the piles (Fig. 6), then it must be distributed by the deck slab acting as a horizontal plate girder along perhaps 27 bents. The calculated bending moment for this condition is over 16,000 tons-ft. which requires 20 sq. in. of steel along the edge of the wharf to resist it. This was provided along the crane beam to the west of the wharf, but since there is now no crane on this beam not all the steel was extra to the original design.

The wharf as a flat slab is calculated to deflect only about 0.06-in. at the point of impact, and the compression of the raker piles to contribute a like amount. The steel along the west side was carried diagonally across the deck at the outer end, more or less as conventional shear reinforcing. A special condition arises towards the end of the wharf as shown in Fig. 6 where an adequate length of deck cannot be involved. For this reason two extra bents of piles, both verticals and rakers, were driven between the piles of the first two end bays. Viewed from a small boat, the cluster of piles at the end looks like the legendary "immovable object" but then a heavy ship plunging about gives the impression of being the "irresistible force."

3.4. Cost of Fendering

The contract cost for one bay or 12-ft. of fendering on the east berth is as follows:

	£	s.	d.
1. Turpentine pile, 70-ft.	144	0	0
2. Pile shoe	4	0	0
3. Shaping pile head	3	10	0
4. Sheet metal cap to pile	1	5	0
5. Cut flats on piles for item 6 ...	4	0	0
6. Vertical chafing strip 12 in. by 8 in.	29	16	0
7. Main waling of three 10 in. by 8 in.	37	10	0
8. Backing pieces of two 14 in. by 10 in.	38	0	0
9. Tarring timbers	4	0	0
10. Bolts to timber: Naval brass ...	31	0	0
11. Bolts to timber: Black	24	3	0
12. Chains and eye bolts	12	0	0
13. Rubber buffers, 2 at £65	130	0	0
14. Fixing above, 2 at £6	12	0	0
15. Spigots to buffers, 4 at 63/6 ...	12	14	0
16. Backing plates to buffers, 2 at 49/6	4	19	0
17. Bolts to buffers	1	7	0
18. Extra reinforcing to deck on west edge	33	0	0
	£527	4	0
Add 15% for overheads, say	78	16	0
Total:	£606	0	0

Per foot run of wharf face, this is £50.10.0.

The energy absorbing value of the system per bent of 12-ft. is approximately:

	Force (ton)	Distance (in.)	Energy (in. ton)
Two buffers 60% compressed	120	16.2	972
Two tyres as tested ...	100	9	94
One Fender Pile as cantilever	0.5	16	4
Total Capacity:			1,070 in. tons

The capacity is therefore about 89 in. tons per foot run of wharf face, and the cost per in. ton is 11s. 3d. The cost of the buffers ex factory was £62 19s. 3d. or on a capacity each of 486 in. tons, the cost per in. ton was 2s. 7d. The buffers alone ex factory are thus about 25% of the total cost of the fender system, but provide far the greater part of its direct energy-absorbing capacity. It could be suggested for application to other work that the cost of the turpentine piles might be reduced by £38 and that the extra steel in item 18 above should not be included, in which case the cost per in. ton would be just under 10s. The rubber in the buffers is 3.18 cu. ft. each, so the cost per cubic foot ex factory was just under £20. By comparison, the helical steel buffers which were originally specified were quoted at £55 ex factory in England for 75 in. ton, making the cost 14s. 8d. per in. ton. On these figures, the cost of rubber as a shock absorber is less than 20% of that of steel springs and housings.

In reviewing the cost of £50 10s. per foot of wharf face and the fact that the energy-absorbing capacity per bent is calculated to be something like twelve times that of the timber wharf (Newton King) which has apparently given satisfactory service for many years, it must be stated that the action of a raker braced concrete wharf is entirely different from that of a cross braced timber one. So much of the fender system would be required, even without the rubber buffers, that their cost is highly justified. In any case, the cost of the fendering is equivalent to only about 9-ft.

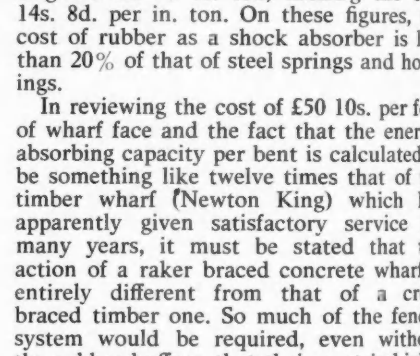


Fig. 7. Section of east side of wharf.

width of the general wharf structure (see later analysis in Part II). Little in his introduction says: "It is doubtful if even the heaviest of maintenance charges could offset the extra initial cost of a gravity structure compared with a piled one, but the fenders of the latter may represent 25% of the total first cost and if the best overall economic efficiency is to be obtained from

Port of Taranaki—continued

open structures it will usually be found that the fendering is the most important feature of the design — especially where water depths in excess of 20-ft. and vessels greater than 10,000 tons are involved." The writer concurs in every respect, particularly for the conditions prevailing at Port Taranaki.

Per inch-ton, however, the cost appears to be low and this may be partly accounted for by the use of rubber in simple compression. A comparison with gravity-type fenders is not quite a direct one, since they may be so spaced—as for tanker dolphins—that the cost should be considered per dolphin rather than per foot run of wharf. However, the cost per inch-ton can be related and the following figures are taken from Little's paper and the ensuing discussion:

	Type	Cost per foot	Inch-ton per foot	Cost per inch-ton
Moturoa	Rubber	£50.10s.	89	11s.3d.
Little's	Rubber & steel pile	£25-£45	10	50s. to 90s.
	Gravity	£50	30	33s.4d.

During the discussion is was stated that gravity fenders generally cost about £3 per inch-ton and that fenders using rubber cost about half this or 30s. per inch-ton. The substance of the paper cannot, of course, be condensed in so brief a summary of costs. The figures above, apart from costs, suggest that the energy-absorbing capacity of the fendering provided for the Moturoa Wharf is relatively high.

3.5. The Fendering in Service

Naturally it will be years before the performance of the fendering can be properly judged. However, the indications so far is encouraging. The first ship to berth was the cruiser "Royalist." She berthed on March 16, 1957, and came in practically parallel with the wharf face at a velocity estimated to be 9-in./sec. Owing to the shape of the ship, the effect was localised on a comparatively short length of fendering, perhaps 50-ft. The maximum buffer deflection was estimated by eye as 7-in. The rebound effect was quite marked, but this was not unexpected since no tractor tyres or soft cushion were provided. It has always been intended to have some such first soft cushion and the present scheme is to use rubber sausages of 4-in. diameter strung along the walings. The impact on the concrete deck was also imperceptible.

Accurate measurements were not taken, but an attempt will be made to analyse the situation. In the 50-ft. of contact the buffer compression would be approximately as under:

No.	Compression	Load	Energy
1	7 in.	26	91
2	6 in.	23	138
2	5 in.	19	95
2	3 in.	11	33
	Total	79 tons	357 in. tons

For a dead weight of ship of 5,700 tons moving at 9-in./sec. the energy is 500 in. tons. On these assumptions, the buffers have accounted for 71% of the energy of the ship, which is a reasonable check against the 50% assumed earlier.

Subsequent reports are that merchant ships are behaving very well when tied up at the eastern berth.

4. General

The overall cost and some details of individual items are given in Appendix 2. The contractors were Downer-Morrison-Knudsen and the Board were fortunate in having them do the work. Acknowledgment is made of the courtesy and assistance given by the chairman, the secretary and the engineer to the Board.

Mr. D. P. Williams, an American was project manager for the joint venturers Downer-Morrison-Knudson over the major portion of the work and the project engineer was T. Hindley, A.M.I.C.E., who later became project manager. The writer was consulting engineer for the project and G. W. Butcher, resident engineer.

Part II Construction and Site Problems (G. W. Butcher) will appear in the October issue.

APPENDIX 1

Pile Properties

Properties	Vertical	Raker
1. Size (in.)	18×18	24×16
2. Area concrete (sq. in.) ...	320	380
3. Number 1 in. dia. steel rods ...	8	14
4. Area Steel: all rods (sq. in.)	6.29	11.00
5. Percentage steel	1.96	2.88
6. Concrete equivalent: $n=$ 10 sq. in.	63	110
7. Total equivalent area concrete (sq. in.)	383	490
8. Moment inertia x-x in.^{4} ...	10,600	25,420
9. Radius gyration x-x in. ...	5.25	7.22
10. As Beam $p' = p =$ ...	0.0086	0.0115
11. As Beam d (in.)	15.2	21.2
12. As Beam bd (in.^{2})	274	340
13. As Beam bd^2 (in.^{3})	4,170	7,200
14. Transverse B.M. on 70-ft. at 1 in 3 rake (k'')	887	1,075
15. M/bd^2	212	149
16. Bending only f_c (lb./sq. in.)...	1,040	560
17. Bending only f_s (lb./sq. in.)...	25,600	14,700
18. L/r x-x for 70-ft.	160	116
19. For $P=220$ ton M/P ...	1.8 in.	2.18 in.
20. For $P+M_{max}$ f_c = lb./sq. in.	1,700	1,250

APPENDIX 2

Properties of Typical Rubber Buffer
Length 27 in. Outside diameter 18 in.
Inside Diameter 8 in.

Note: Stress given is on original cross-section. Load-compression ratio constant to 55% compression.

Stress (lb./sq. in.)	Load (tons)	Compression (%)	Compression (in.)	Energy (in.-tons)
110	10	10	2.7	13.5
220	20	20	5.4	54
330	30	30	8.1	121
440	40	40	10.8	216
550	50	50	13.5	338
605	55	55	14.85	408
692	63	57.5	15.525	448
825*	75	60	16.2	494

*On final diameter (see Fig. 4) stress at mid-height is 260 lb./sq. in.

APPENDIX 3

Costs

	Total Cost	Cost per sq. ft. of Wharf	% of Total
1. Piles and piling ...	241,744	2.84	40.2
2. Deck slab and beams	126,936	1.49	21.1
3. Fender system ...	63,800	0.75	10.6
4. Timber end (50-ft.)	24,922	0.29	4.1
5. Bollards	17,031	0.20	2.8
6. Windbreak ...	18,812	0.22	3.1
7. Rail and crane track	77,107	0.91	12.8
8. Water mains ...	11,129	0.13	1.8
9. Miscellaneous ...	21,003	0.26	3.5
10. Total cost	£602,484	£7.09	

A New Technique for Driving Long Precast Concrete Piles

There are many problems associated with the transporting, casting, pitching and driving of piles in excess of 70-ft. in length. In view of this, prestressed concrete offers the advantage that piles may be reduced in section for a given length where the dimensions are otherwise determined by the moment of resistance necessary to ensure pitching without damage. Furthermore, any tension crack which may develop during slinging will close under the prestress, when the pile is in the vertical position and the concrete (in properly made piles) continues in a state of permanent precompression eminently desirable in a pile subject to tensile driving stresses.

These advantages can be considerably augmented if the length of the pile can be increased by a series of elements during the driving operation, provided that the prestress can be continuous throughout the entire length. Much smaller piling equipment would be needed for the short lengths and the size of the cranes and other tackle for handling the individual elements would be correspondingly reduced. Furthermore, the ability to determine the total length of pile after final penetration has been reached entirely eliminates the risk of either surplus pile length or too short a pile, both of which are hazards in driving even normal length piles. A simple means of achieving this already exists with the Macalloy coupler, but two important conditions must be satisfied. A means must be provided to disperse the concentration of stress which would arise from high points in contact on two imperfectly matched abutting surfaces, and the joint must be made quickly and cheaply with a material having the similar mechanical properties to concrete.

A type of resin is now available which appeared to fulfil these requirements, and, in order to test its efficiency in the field, it was arranged for an experimental pile to be driven on a construction site at the

Precast Piles—continued

South Eastern Gas Board's Wandsworth Works. This was carried out on behalf of McCalls Macalloy Ltd., by the Demolition and Construction Co. Ltd., who were at that time working on the site, driving 16-in. x 16-in. x 50-ft. long prestressed piles.

Fig. 1. Head of first section after driving showing couplers.

Fig. 4. The completed joint.

The experimental pile was precast in three sections, the first section 25-ft. in length 16-in. x 16-in. was fully prestressed by four $\frac{7}{8}$-in. diameter Macalloy bars. At one end of this the bars terminated in couplers and on the other end the anchorages were cast in and a precast pile shoe provided. This section was hoisted and driven until the head was at a convenient working height above the ground (approx. 3-ft.).

A second 25-ft. length of similar cross section, partially prestressed to allow it to be handled, was suspended vertically in the guides above the first section. The temporary prestress was released and the ends of the bars screwed into the couplers at the head of the lower section.

The resin was delivered as a kit of finely ground powder and a syrup-like liquid in separate containers, and these were mixed together to a putty-like consistency. The mixture was then spread over the joint and the second section was lowered into place down the prestressing bars, until the weight of the second piece of concrete squeezed the excess material out from the sides of the joint. Immediately this operation was completed, preparations were made at the top of the pile to commence jacking and by the time this had been completed, a matter of about 15 minutes, the joint was hard enough to transmit the prestressing forces which were then applied to the top of the pile. The completion of the jointing and stressing operation occupied in all 37 minutes.

Driving was recommenced at once and

Fig. 2. Second section being lowered onto first section.

while the joint remained above ground, no movement of any kind or sign of distress was apparent. Shortly after the joint had been driven out of sight, a further short piece 5-ft. long was connected to the top of the second section by an exactly similar procedure to enable the joint to be kept under observation during driving. Driving then recommenced until full depth was reached. For both top sections a special cruciform M.S. anvil block was used under the driving helmet to accommodate the projecting stressing bars.

Although the driving was not exceptionally hard ($\frac{1}{4}$-in. per blow set with 4-ton hammer falling 4-ft.), the behaviour of the joint inspired every confidence that it would withstand considerably more severe conditions.

According to the manufacturers, the hardened resin has the ability to resist attack from salt water and even the most corrosive soil in industrial areas. The compressive strength has been demonstrated to be in the region of 11,000 p.s.i. and the tensile strength in the order of 1,300 p.s.i. Its adhesion to concrete is such that when a series of standard brickettes were broken, jointed by the resin and re-broken, the fracture passed partly through the jointing material and partly through the concrete.

Since completing the above experiment a further spun concrete pile of cylindrical shape, 18-in. diameter and 4-in. wall has been driven successfully. A 15-ft. section was jointed in situ to two other 15-ft. sections which had been previously jointed and prestressed at the maker's works. Again no damage occurred to the jointing material in either of the two joints. The driving in the latter stages was moderately severe.

Fig. 3. The joint: bars being coupled.

Fig. 5. The head of the second section after stressing.

The resin, known as "Artrite 13-10," is available in three grades for use in cold, medium or hot weather respectively, the hardening time of the putty being to some extent dependent on the ambient temperature. The hardening time can be arranged to be as short as ten minutes but generally between half and two hours is most commonly desired.

Proposed Ore Transit Port for Greenland

It has recently been announced in Copenhagen that the Danish Government has accepted in principle a proposal made by the Canadian Ungava Iron Ores Company to build a transit harbour in Greenland for the export of iron ore from Canada. The harbour would be used to stockpile ore for markets in Europe and the United States, which would be a great advantage to the Company as Canadian harbours for this export trade are icebound for nine months of the year, whereas the projected port would be open most of the year.

The installation would consist of berths and an ore handling plant where vessels of up to 45,000 tons d.w. could be unloaded and loaded.

Marine Site Investigations

Methods of Boring, Sampling and Rock Drilling

By R. F. HEYNES, B.Sc. (Eng.), A.M.I.C.E.

ONE of the first phases of any civil engineering construction scheme should be the investigation of ground conditions. The objects of such an investigation are to assess the suitability of the site for the proposed scheme, and to provide data from which adequate and economical foundations may be designed. This type of investigation is normally referred to as a Site Investigation, and the recommended procedures for conducting such investigations are set out in the British Standard Code of Practice CP2001 (1957).

Site investigations for construction schemes in rivers, docks and harbours or for other maritime works are usually referred to as marine site investigations, and the information required from them is similar to that needed for a construction scheme on land. There are, however, special problems brought about by the need to conduct the investigation over water, and these are described in this article. Boring, sampling and in-situ testing procedures, which are common to both land and marine site investigations, are also described.

General Procedure

The general procedure for both land and marine investigations is to sink boreholes into the ground, and to take samples of the soil at intervals of depth. Laboratory tests are carried out on specimens taken from cohesive soils, and the bearing capacity and compressibility of the soil at various depths is assessed. In certain strata in-situ field tests are conducted and similar information is deduced from the results. The information thus obtained assists in the economic design of the foundations of the structure under consideration.

The number of boreholes depends on the area of the proposed works, but in the more difficult investigations the cost sometimes dictates their scope. Boreholes are generally sunk to a depth at which it is estimated that no significant stressing of the soil will result from the load imposed by the completed works. If the boreholes are required only to determine dredging conditions they need only be sunk to final dredging level or a little deeper, and laboratory testing may not be necessary.

When rock occurs within the depth under investigation, cores of the rock are taken for examination and testing.

Types of Boring

1. **Shell and auger.** The equipment required for this type of boring is basically a light derrick, a powered winch unit, boring tools and lining tubes. Rope or wire bond carrying a swivel rod and drill rods is led from the winch drum over the pulley at the top of the derrick. A heavy boring tool screwed to the lowest rod is raised and lowered continuously. For sand and gravel the boring tool is a heavy metal cylinder with cutting edge and flap valve and is called a "shell." In clays a cylinder with a cutting edge is used. Some gravels and clays may also be penetrated by augers screwed to the drill rods and manually rotated. A heavy solid tool with sharpened edge, called a "chisel," is used to penetrate soft rocks. Fig. 1 shows the derrick of a boring rig being erected on a cantilevered platform on the quayside at Swansea.

2. **Wash boring.** A wash pipe of 1-in. to 2-in. diameter is used in conjunction with a slightly larger diameter casing (2½-in.—4-in.). Water under limited pressure is passed down the wash pipe, and soil particles are washed from the bottom of the wash pipe through the small annular space between it and the casing. The water velocity in the annular space is greater than that in the wash pipe, and the soil particles are carried upwards without undue difficulty. The apparatus shown in Fig. 2 and a light derrick and winch are required.

When gravels or stiff soils are encountered a chopping bit is attached to the wash pipe, and the pipe is worked up and down and turned at the same time. This usually breaks up the material into fragments.

Undisturbed samples of soil 1½-in. in diameter can be taken in cohesive soils by this method, and a split-spoon sampler is used to recover sand samples for in-situ density determinations by the "standard penetration test" method, to be described later.

Wash borings have the advantage of speed and economy in favourable conditions, but tend to be less accurate and limited in the information provided than the shell and auger method, unless experienced operators are used.

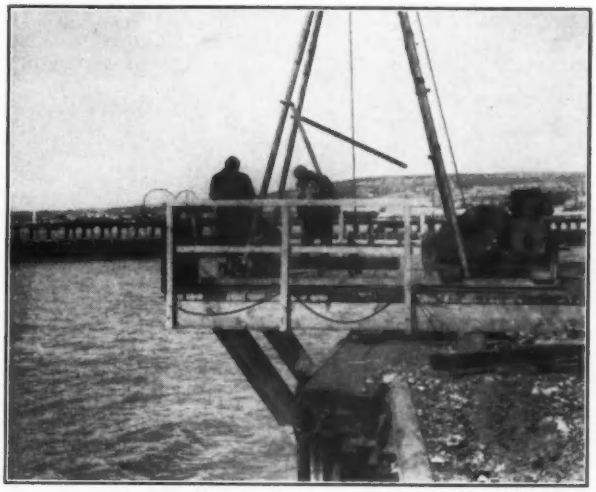


Fig. 1. Shell and Auger boring rig on cantilevered platform. Derrick being erected.

3. **Jetted probings.** When preparing a dredging scheme it is often necessary to find the depth below sea bed level to a hard stratum. A rapid means of providing this information over a large area is by use of jetted probings.

Jetted probings are made by applying a high pressure water jet through a 2-in. or 2½-in. diameter pipe and washing away the soil beneath the pipe. As the soil is washed away the pipe is allowed to follow, and penetration continues until a hard stratum is reached.

During penetration through soft overburden an experienced operator can tell the difference in the consistency of the soil being penetrated by the "feel" of the pipe when it is being worked up and down. It is sometimes necessary to increase the water pressure in order to achieve penetration, and this is also a measure of the consistency of the soil. It is possible to obtain soil samples from within the pipe, but these samples would be much disturbed by the effect of the water jet. A light derrick and winch unit are used, as for wash borings.

4. **Rock core drilling.** For obtaining rock cores a rotary type rock coring drill is used. Special coring bits containing industrial diamonds are rotated into the rock, a water flush being used to wash the chippings clear. Downward pressure is applied to the drill rods by a hydraulic system or by a screw feed, while rotation is provided from the drill engine.

In this type of drill, the drill rods are secured by chuck jaws to the drill head, which in turn is fixed to the drill frame. Consequently the drill in its normal state may be used only from a fixed platform. In order to use the drill from a floating craft a special technique is adopted, and this has been successfully employed

Marine Site Investigation—*continued*

on a number of investigations with which the author has been concerned. Even so the movement of a craft causes heavy wear on the drill rods and drill head.

The above types of boring can be satisfactorily conducted from floating craft as well as on land and from fixed platforms, although rock core drilling may only be done from a craft in reasonably calm water.

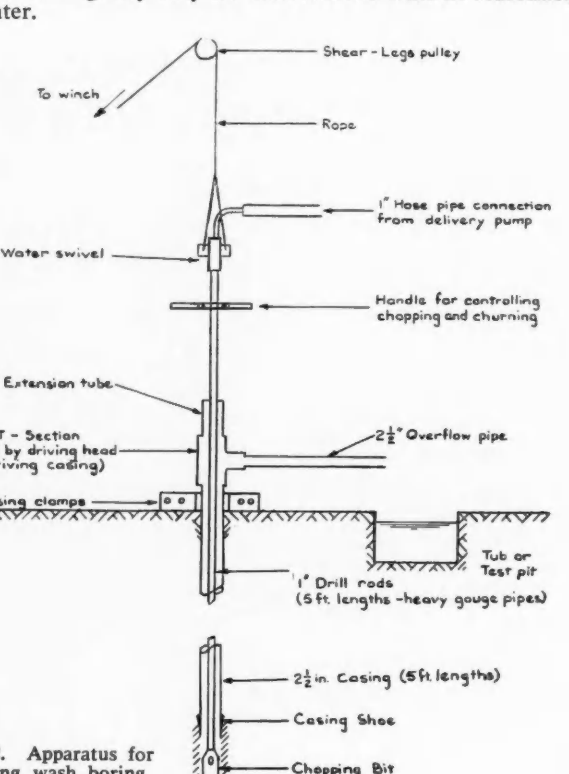


Fig. 2. Apparatus for making wash boring.

Sampling and In-situ Testing

1. **Undisturbed samples.** When shear strength or consolidation tests on clays or silts are contemplated, undisturbed samples of the soil must be obtained. To obtain an undisturbed sample a steel cylinder 1½-in. or 4-in. in diameter (depending on the test required) is carefully forced or lightly driven into the soil. Care must be taken to avoid overdriving the sample and thereby compressing it. In order to prevent the sample from drying its ends are sealed with wax.

Great care must be taken to obtain undisturbed samples because any disturbance of the soil will lead to unreliable strength test results. For the same reason it is essential to prevent a change in moisture content of the sample. When the clay is soft it is sometimes necessary to join three sample tubes together in order to recover a sample.

2. **Disturbed samples.** For certain soil tests on granular soils a sample in its disturbed state as recovered from the boring tool is satisfactory. When such samples are used care must be taken to retain the fine material in order to ensure that the sample is representative.

3. **Water samples.** Samples of ground water are taken for chemical analysis, and for checking the presence of substances which may be harmful to concrete or structural steelwork. It is often difficult to obtain ground water samples in marine borings owing to the seepage of sea water into the boreholes. It is advisable to take samples of sea water at various states of the tide for chemical analysis in connection with its attack on marine structures.

4. **Bishop sampler.** This apparatus was designed by Dr. A. W. Bishop of Imperial College[1] to obtain undisturbed samples of silt and sand, below ground water level if necessary. Dry density and particle size distribution tests may be conducted on the samples, while visual examination can provide information on laminations and other important factors.

The sample tube is lowered to the sampling depth inside an air reservoir, from which the water is then expelled by air pressure. The air pressure inside the reservoir is continually applied until the sample is brought to ground surface. This apparatus has been successfully used by the author's firm on marine site investigations.

5. **Standard penetration test.** This in-situ test is used in sand and fine gravel to obtain an indication of the bearing capacity of the soil, and to determine its in-situ density.

An open-ended sampler is forced into the soil under standard conditions. A hammer 140 lb. in weight is dropped 30-in. on to a drive head connected to the sampler. The number of blows of the hammer required to drive the sampler 12-in. into the soil is recorded. The bearing capacity of the soil is obtained from a conversion table.

In medium gravel a solid cone is used instead of an open-ended sampler[2].

Because of its simplicity and speed of operation, the standard penetration test is admirably suited to marine site investigations, and, as mentioned earlier this test can be conducted in conjunction with wash borings.

6. **Vane test.** The vane test is an in-situ test used in soft silts and clays. A four-bladed vane attached to a string of rods is lowered to the required level. A torque is applied at the top of the rods, and is increased until the rods begin to rotate. The torque required to cause rotation of the rods (and therefore the vane)) is recorded. This torque is a function of the shear resistance of the soil at the vane, and a simple conversion gives the latter value.

Some difficulties arise in using the vane test apparatus from floating craft, and special modifications to the apparatus are necessary.

7. **"Dutch cone" test.** The apparatus for this test is designed to push a steel cone into the soil to a maximum depth of about 120-ft. This is done by hand winching, the apparatus being secured firmly to the ground. The gearing of the winches permits a total downward force of 10 tons to be exerted.

The resistance of the soil to the cone is recorded, as is the skin friction on the driving tube. The apparatus includes a soil sampling kit which obtains undisturbed samples of the soil.

The resistance of the soil to the cone provides an indication of the shear strength of the soil the values of which can be obtained from a conversion table.

The Dutch cone test apparatus, also known as the Deep Sounding apparatus, is usually used in conjunction with normal borings. It provides a quick method of supplementing the information obtained from borings, but it is necessary for a correlation to be established initially.

SPECIAL PROCEDURE FOR MARINE SITE INVESTIGATIONS

The types of boring and drilling described above are common to both land and marine site investigations, as are the sampling and in-situ testing methods.

It is, however, necessary to make special arrangements for carrying out a site investigation over water, and the intelligent choice of method is an important factor in the success of the whole operation.

Setting out

When borehole positions are sited near existing jetties or roadways it is usually a simple matter to set out the correct positions. Distances can be measured by tape, and directions given by theodolite readings.

Setting out borehole positions offshore but within easy sight of land may be done in several ways, and the procedure adopted on a recent contract with which the author was concerned was as follows.

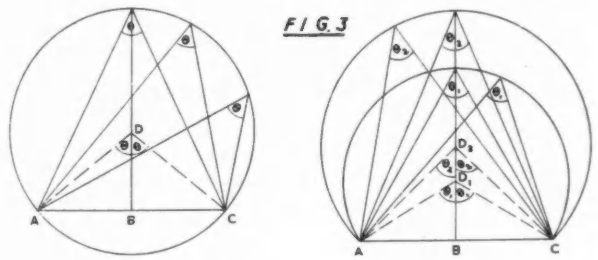
The site investigation was for a marine terminal whose approach leg was to be about 3,000-ft. long, with a pier head about 2,600-ft. long. Boreholes were located along the centre lines of approach and head. About 6 shore survey stations were

Marine Site Investigations—continued

selected, clearly marked and accurately surveyed, their correct positions being plotted on a chart kept on the boring vessel. The approximate position of a borehole was obtained by sextant readings taken from a launch on to several pairs of shore survey stations. A marker buoy was then dropped, and its exact position determined by further sextant readings on to several pairs of shore survey stations. The position of the buoy was plotted on the boring vessel chart, and its location relative to the borehole position was thereby shown. The boring vessel was then moved to the estimated position of the borehole and anchors were laid. A number of sextant readings was taken from the boring platform and the anchor cables were adjusted until the vessel was in the correct position. Finally, a check was made later on the borehole position by theodolite readings on the borehole casing from at least two shore survey stations. Although several operations were involved the total time taken to bring the vessel from one borehole position to the next rarely exceeded three hours, and more than half this time was used in manœuvring the vessel and recovering and laying anchors.

Another method of setting out is the use of a Circle Chart[3]. This makes use of the fact that the angle subtended at the circumference of a circle by a chord is equal to half the angle subtended at the centre of the circle by the chord. Several land base lines are accurately surveyed and plotted on a chart on the boring vessel. The radii corresponding to various sextant angles subtended by the base line are then calculated, and a series of circles drawn as shown in Fig. 3. The procedure is repeated for other base lines and the result is as shown in Fig. 4.

Once the Circle Chart is completed the operation of setting out the boring vessel over a borehole position is achieved very quickly. The vessel's position is first determined. A sextant reading is taken on to the survey stations of a convenient base line. By interpolation, an arc representing the sextant angle can be sketched on the Circle Chart. This procedure is repeated using an adjacent base line. The intersection of the two arcs thus sketched on the Circle Chart will give the position of the



RADIUS AD = AB cosec θ, where AB is half length of base line.

LOCUS OF POINTS DESCRIBED BY ANGLE θ

LOCI OF POINTS DESCRIBED BY ANGLES $θ_1$ AND $θ_2$

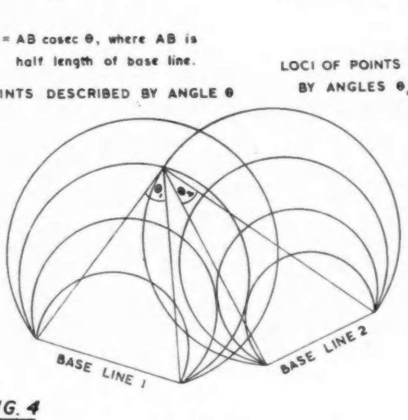


FIG. 4

POSITION OF BOREHOLE (OR VESSEL) GIVEN BY SEXTANT ANGLES $θ_1$ TO BASE LINE 1 AND $θ_2$ TO BASE LINE 2

vessel. The vessel is then moved to the estimated position of the borehole, and two further sextant readings taken. This procedure is repeated until the vessel is in the correct position.

The Circle Chart method is most useful when many borehole positions have to be located; then the speed of setting out warrants the time taken in plotting the Circle Chart. Much time is saved by avoiding the use of station pointers in plotting positions on the chart.

The Circle Chart method proved most useful on a marine site investigation undertaken by the author's firm in 1956 at Kuwait.

Fig. 5. Boring rig operating from tubular scaffolding staging.

The investigation included nearly 200 shallow boreholes sunk in connection with a dredging scheme. The progress of the boring depended to a great extent on the speed of setting out, and the author can speak from first hand knowledge of the success of the Circle Chart method on this occasion.

When the area under consideration is out of sight of land, and shore survey stations cannot be used, it is necessary to erect survey towers near the site by using large diameter tubes which are sunk into the sea bed and project above sea level.

Stagings

Stagings constructed of 2-in. diameter tubular scaffolding are the simplest method to use for borings when the depth of water at high tide does not exceeed about 25-ft. If the distance offshore does not exceed about 150-ft. it is usually most economical to construct a catwalk to a platform about 15-ft. by 20-ft. The catwalk must be of sufficient width to allow the boring equipment to be moved along it, that is, about 5 to 6-ft. The staging must be properly braced, and if this is done it will withstand fairly rough seas owing to the small area of resistance presented by the scaffold tubes. The deck level of the platform and catwalk must be at least 2-ft. above the highest sea level anticipated, and gaps should be left in the decking to allow an abnormally high sea to rush through rather than strike the underside of the decking with its full force.

Stagings are cheap to erect as the scaffolding can be hired, and a few experienced operators can build a staging speedily. Increasing use is now being made of prefabricated trusses or " self lock " sections as these help to increase the speed of erection and dismantling. A good combination is to use conventional scaffolding as the base of the staging, and in this way overcome the slope of the sea or river bed; the prefabricated sections can then be used in the superstructure.

The main advantage of stagings, apart from low cost, is that boring may continue in almost all states of weather and in all tidal conditions.

In harbours and rivers a catwalk and platform may present a hazard to shipping which may not be acceptable. In any case great care must be taken to have the staging marked and lit in

Marine Site Investigations—continued

accordance with the port or river authority's requirements. Fig. 5 shows a boring rig operating on a tubular scaffolding staging erected in the River Thames at Millbank. In this case a short catwalk only was required.

Boring towers

When the distance offshore is too great for the use of a catwalk, or the water is too deep, the usual alternatives are a boring tower

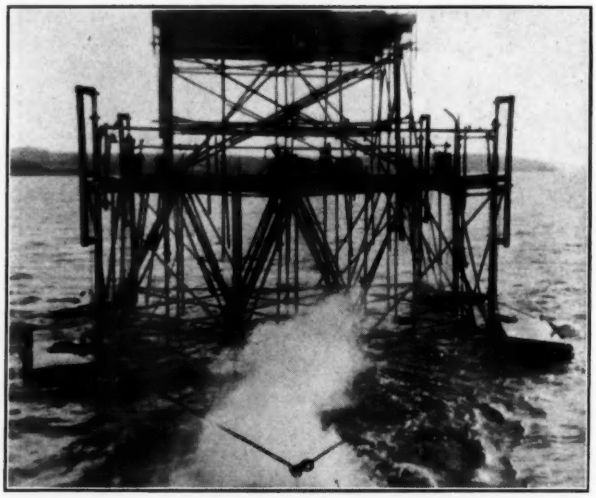


Fig. 6. Tubular scaffolding boring tower being towed into position by launch. Boring equipment is lashed to top platform.

or a floating craft. The boring tower consists of a platform supported by a braced framework the main members of which are the four corner legs. There are several variations in the construction and operation of this type of tower, but the author has had experience of a tower made of 2-in. diameter alloy tubing with main legs about 4½-in. in diameter. This tower was designed to be floated in position thus avoiding the use of a crane barge. A block and tackle arrangement at each corner of the platform permitted it to be levelled. The tower could be used in a maximum of about 35-ft. of water, and the boring rig and equipment were placed on the platform before the tower was towed to site. Fig. 6 shows the tower being towed into position by a launch, using four Admiralty tank floats to give the necessary buoyancy.

Another type of tower consists of structural steel prefabricated trusses which are lowered into position from a craft, and assembled when tidal conditions permit. This method tends to be rather costly, but it has the advantage of being suitable for use in greater depths of water than the "floating" tower mentioned above. The floating crane used to assemble and move this type of tower can only work in reasonably calm conditions.

Boring towers must be securely guyed with wire ropes to anchors or concrete clumps. It is essential to have a boat or launch alongside at all times.

Although boring from a tower can be carried out in almost any sea conditions, the sea must be calm enough to enable the operators to land on the tower from a launch. The operators must also be taken off the tower if bad weather threatens, before the sea becomes too rough.

Platform supported on large diameter casing

Sometimes it is better to drill from a small platform fixed to a column of large diameter (about 15-in.) casing drilled into the sea bed. The conditions in which this method might be preferable are when bedrock proving by rock coring is required, and (a) there is a large tidal range and deep water; (b) sea conditions make it difficult to drill from a floating craft.

The large diameter casing is driven well into the sea bed by using shell and auger boring methods from a floating craft. A supporting unit is screwed to the top of the casing. The unit consists of three or four supporting arms for the platform. The platform must be as small as possible, and the method is confined to the use of small petrol or diesel drills or medium air driven drills.

The casing must be well guyed to anchors or clumps. This method is best suited to a thick layer of a firm stratum such as gravel or soft rock in which the casing may be securely embedded.

The arrangement shown in Fig. 7 shows an air operated rock coring drill erected on a small circular platform fixed to a 15-in. diameter casing. Air was supplied from a compressor on the main boring vessel. Choppy sea conditions made it impracticable to use the drill on the boring platform erected over the vessel's side. This work, carried out at a site some 50 miles offshore in the southern Persian Gulf, was done for Abu Dhabi Marine Areas Ltd.

Floating Craft

The use of floating craft is essential for investigations in water too deep for the safe or economical employment of the foregoing methods, and for open sea conditions.

For jetted probings and wash borings a suitable craft in fairly calm conditions is a large launch or a small powered barge. The main requirements of such a craft for use on jetted probings or wash borings are: (a) sufficient space for the equipment comprising a pump and wash pipes, and a boring crew of 2 or 3 men; (b) space for an overhanging platform about 4-ft. by 3-ft. amidships or at the bows; (c) a light derrick for use in handling the wash pipes, or the space for a derrick and winch unit to be erected near the platform.

A jetted probing or wash boring may take only a few minutes to complete in favourable conditions, and a small amount of movement can be accepted. Consequently mooring arrangements should be simple, anchors at bows and stern usually being sufficient. These would be laid and recovered by the craft itself.

The type of craft used for a full scale investigation (including jetted probings and wash borings) can vary from a dumb barge to

Fig. 7. Air operated rock coring drill on platform supported by 15-in. dia. casing drilled into sea bed.

a large powered vessel providing accommodation for the boring crew. The choice of craft is governed by the number of borings, sea conditions, location of investigation, and availability of craft in the area.

Dumb barges are usually the cheapest to hire, but their use must be confined to inshore work, and in locations where the

Marine Site Investigations—continued

water is not rough. Fitting out costs are fairly high, as additional winches have to be fitted, and the hold must be boarded over and covered with tarpaulin. It is usual to place ballast in a dumb barge to increase its stability in the water and reduce the area of hull exposed to wind. A small tug or large launch is required to lay and recover the barge's anchors, and to be in attendance in case of bad weather. Pontoons may in some cases be suitable for this type of work, and they have the advantage of reasonably good mooring arrangements.

Fig. 8 shows a piling pontoon used as a boring craft by the author's firm at Kuwait. The pontoon had to be towed into position when long distances were involved, but had the advantage that its piling frame and winch could be used to lift a boring tower into position, as shown in Fig. 9.

Many kinds of powered craft have been used successfully for marine site investigations. The author has had personal experience of a variety of craft including a Mersey motor barge, a salvage vessel of 220 tons, a L.C.T., and a 1,230-ton freighter, the last two having been used in the Persian Gulf, where it was necessary to accommodate boring crews on board. Generally, this type of craft needs less preparation than a dumb barge to suit it to site investigation work. Costs are higher of course, and as the size of the craft increases it becomes necessary for the boring crews to work double shifts in order to offset the high hire charges for the vessel. The larger vessels are able to lay and recover their bow and stern anchors which speeds up the operations of mooring and moving between borehole positions.

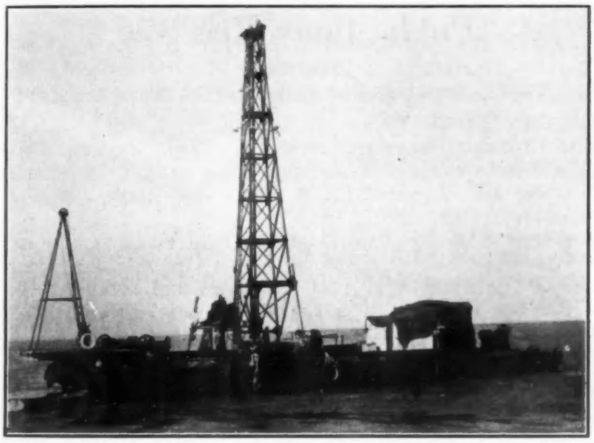


Fig. 8. Piling pontoon in use as boring craft.

The usual method of boring from the above types of craft is to operate from a platform overhanging amidships or at the bows. The platform needs to be about 16-ft. wide with an overhang of about 6-ft. The author prefers the platform to be amidships and near a hatch. It is then possible to make the fullest use of the vessel's derrick, and to use the hatch coaming as a support for the platform. The platform bearers may be laid across the top of the hatch coaming, bolts being used to secure the bearers to the underside of the coaming. The platform should be stoutly constructed as it has to withstand a severe buffeting from the borehole lining tubes in choppy seas. The bearers are also propped near the gunwhale. Timber decking is laid across the bearers, a slot about 2-ft. wide being left to accommodate the borehole casing.

Fig. 10 shows a boring platform erected over the side of the boring vessel at a location some 20 miles offshore in the north Persian Gulf.

It is essential to keep the movement of the vessel to a minimum during boring operations, particularly when diamond drilling. Consequently at least six anchors of suitable weight should be used, a suitable arrangement being to have bow and stern anchors, and an anchor at each quarter. The number of anchors may be reduced in sheltered conditions, but the number should not be less than four.

The procedure for laying anchors must depend on the direction of the prevailing wind and tidal currents, and the relative effect of each on the vessel. The object when positioning the vessel is to minimize the effect of wind and tide. A common practice for positioning a powered craft is to steam against the current towards the borehole position, dropping the stern anchor while still short of it. The vessel then overruns the borehole position, and the bow anchor is dropped. The vessel is then allowed to move back with the current, paying out the bow anchor cable and winding in the stern anchor cable. Both cables are wound in when near the borehole position. The quarter anchors are laid by a launch and the final adjustment of position is made by use of any of the six anchor cables.

Fig. 9. Boring tower after being lifted into position by piling pontoon.

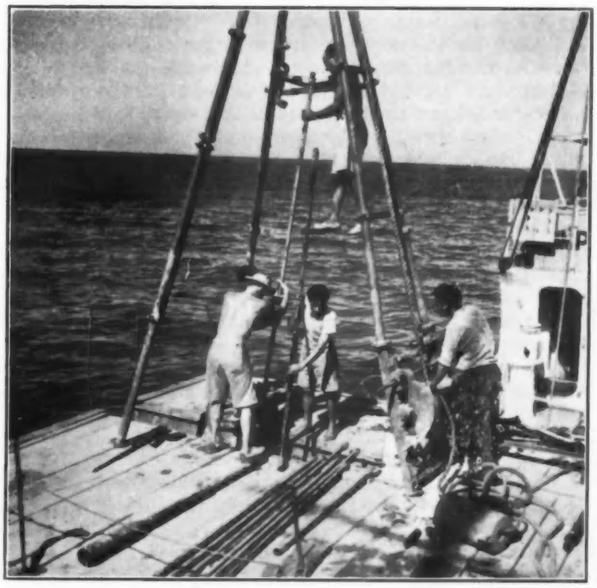


Fig. 10. Boring platform erected over side of boring vessel.

Marine Site Investigations—continued

Test Piling

It is not always possible to estimate with sufficient accuracy the carrying capacity of piles from the results obtained from borings and laboratory tests on soil samples. Often the only satisfactory way to obtain such information is to drive test piles and to subject them to loading tests. As in the case of borings, the cost of test piling increases considerably when a piling craft has to be used.

Fig. 11. Driving No. 4 Rendhex pile from vessel, using 10B3 McKiernan-Terry double acting piling hammer.

The author was recently concerned with test piling work which formed part of a marine site investigation for the British Petroleum Company's tanker terminal at Milford Haven, and for which the boring vessel (a salvage vessel of 220 tons) was adapted for test piling. The boring platform was removed, and a framework of timber baulks was constructed to provide a gate over the ship's side for No. 4 Rendhex piles. The framework was anchored securely to additional baulks wedged into position in the hold. A No. 10B3 McKiernan-Terry double acting hammer and leg guides were suspended from the ship's derrick, and steam pressure was supplied by the ship's boiler.

An interesting feature of the operation was that the longest pile used (95-ft.) was almost as long as the vessel itself, and this pile was taken to the site secured to the side of the vessel. On completion of the driving tests, each pile was secured to the vessel on a rising tide to extract it, although in one case this method was not satisfactory, and a diver was employed to cut the pile at sea bed level. Fig. 11 shows the general arrangement of the piling equipment described above.

The simplicity of the piling gate arrangement was due to the double acting hammer used. However, the use of this type of hammer precludes the application of the Hiley or other dynamic pile formula in calculating the driving resistance of a pile. In the case described above the object of the test piling was to determine minimum penetration below sea bed level and to conduct horizontal pulling tests on pairs of piles. If dynamic formulae are to be applied it is desirable to use a drop hammer or a single acting hammer. Both these types of hammer require a frame or leaders, and the arrangement for supporting a single acting hammer by a cage bolted to a pile is cumbersome.

Dynamic pile formulae for calculating carrying capacity are considered by some people to be of doubtful value in most circumstances. The ideal procedure is to conduct loading tests to failure on the driven piles. Loading tests over water are very expensive however, since it is necessary to construct an independent piled staging to carry the kentledge load. Such loading tests are usually restricted to large scale projects where the cost can be justified by a saving in the number or lengths of piles made by accurate foreknowledge of carrying capacity.

Conclusions

Most maritime construction schemes of any magnitude cannot be undertaken satisfactorily without the preliminary data which a site investigation provides. The value of the information provided by such an investigation is to some extent dependent upon the amount of money available, but much depends on the care with which the investigation is planned before boring commences.

The author thanks his colleague, Mr. M. J. Tomlinson, A.M.I.C.E., Chief Engineer of the Wimpey Central Laboratory, for his advice and help in the preparation of this paper.

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Publications Received

Timber Development Association Annual Report

The 24th Annual Report of the Timber Development Association Ltd. was published recently. This Report, which is available free-of-charge from the Association's Headquarters, 21, College Hill, London, E.C.4, covers the whole range of the Association's work during 1957.

One section of the Report, dealing with research activities at the Association's laboratories at Tylers Green, Bucks, includes a note of studies on shell roof structures and details of the design for the Royal Wilton Carpet Factory—the world's first multiple hyperbolic paraboloid roof in timber — as well as an account of developments in non-destructive testing of timbers by means of ultrasonic pulse velocity measurements, tests on moisture retardant treatments and many others.

Other sections deal with the growth of the T.D.A. Approved Manufacturers scheme, increased activity in the regions, T.D.A. educational facilities, publicity and Press relations and the growth in the volume of work handled by the Association's advisory and design services. These two departments, together with the Regional Officers, handled a total of 16,739 enquiries during the past year.

Since January of this year, when the Association took over the advisory work formerly undertaken by the Forest Products Research Laboratory at Princes Risborough, the volume of enquiries has increased by 30 per cent. All advisory work is handled at the Headquarters of the Association or by the appropriate Regional Officer and enquiries should be addressed to one of these offices.

Recent Advances in Dredges

The increasing versatility of low-cost portable hydraulic pipe line dredges is discussed in a new 12-page bulletin, No. 925, obtainable from the Ellicott Machine Corp., 1611, Bush Street, Baltimore, Maryland, U.S.A.

Thirty-eight photographs show the Ellicott "Dragon" Model dredges in operation on a variety of projects throughout the world. Portability, size, and factors in the engineering and design of Dragon dredges that have helped to increase the scope of hydraulic dredging are explained. In all 15 different applications of these dredges are illustrated.

Economy is discussed, particularly in connection with the dredge's ability to excavate, convey and redeposit water-bound solids in one operation. A chart gives performance figures on six Dragon dredges ranging in size from 8-in. to 20-in. Special auxiliary equipment is also pictured.

Precast Prestressed Concrete for Deck Construction

A note on some Current Investigation Work

(Specially Contributed)

At the present time much attention is being devoted to possible methods of constructing bridge decks in prestressed concrete. Although there have been a number of cases in which precast prestressed concrete bridges have proved to be more expensive than orthodox in-situ concrete bridges, it is now generally accepted that for other than very short spans, prestressed bridge construction is economically attractive, particularly if attention is devoted to the following matters:

(a) The use of a design avoiding intricate and expensive on-site operations.
(b) The employment of a contractor experienced in carrying out the site operations required.
(c) The co-ordination of design, manufacture and erection into one integrated technique.

As for bridge construction, so for quay and jetty work, prestressed (normally pre-tensioned) decking is economically sound, particularly where its use on longer spans allows a smaller number of pile bents to be used.

Fundamental design strength investigations into highway bridge deck loading stresses is presently being carried out under a Ministry of Transport and Civil Aviation programme. Actual stress distributions within certain solid types of beam and slab deck construction have recently been investigated at Lehigh University, U.S.A., and give an indication of the effect of varying the relative amount of cross-stressing.

Similar testing on hollow precast prestressed concrete bridge beam units, with emphasis upon the practical economics of construction, was commenced some weeks ago by a leading British firm of precast concrete component manufacturers, and an opportunity was given for this work to be inspected. The work included the derivation of cost data for different types of unit, this taking the form of planned method study, time and work study and cost study of all on-site operations for five different types of bridge construction. The individual operations included methods of carrying out seven different systems of post-tensioning, including the forming of ducts and the grouting of post-tensioned cables, the formation of diaphragms using both precast and in-situ concrete, on-site shuttering, cross-stressing and the production of post-tensioned beams cast in one unit or cast in sections.

The accompanying illustration shows a corner of the bridge unit demonstration site with, in the foreground, a beam and slab type bridge with the main longitudinal prestressed precast beams resting on an arbitrary abutment. In the background, left, can be seen composite units of inverted-T section, with solid in-situ decking, partially cast. A point of interest in connection with the rectangular beam construction is that both precast and in-situ stiffening diaphragms are under test, each being equipped with ducts for subsequent post-tensioning to provide the necessary transverse prestress. The results of such investigations as those briefly noted here are of particular value in enabling the designer to draw up an economic prestressed structure.

Given reliable cost data, accurate cost comparisons between alternative methods of concrete bridging become possible. Although such design is of course limited by the maximum manufacturing and transporting capacity of manufacturers specialising in precast unit construction, it is clear that pre-tensioning will generally be cheaper than post-tensioning, and that for medium spans of up to about 75-ft., the former method is normally economic vis-a-vis ordinary untensioned reinforcing. For spans of between about 75-ft. to 125-ft., post-tensioned girder bridges tend to be more economic, provided the design does not call for complicated and unrealistic site work.

The results of current research programmes may lead to the amendment of present practice in bridge and jetty deck design. The former now normally follows the recommendations of Ministry of Transport Memorandum No. 577 on Bridge Design and Construction. In the case of highway bridges, Ministry of Transport and Civil Aviation requirement in Britain covers two classes of traffic, which produce "normal" and "abnormal" loading. Generally these loadings correspond to 30 units and 45 units respectively of a standard loading train as defined in British Standard 153, Part 3A. The "normal" loading corresponds generally to a live distributed load of 220 lbs. per square foot plus a knife-edge load of 2,700 lbs. per foot width. Where quay or jetty decking is concerned, design is usually based upon the maximum live loading to be allowed on the structure—usually the relatively heavy loading of dock-side craneage plus a distributed live loading of between 2 and 6 cwts. per square foot, according to the type of road vehicle, mobile crane or transitory cargo anticipated.

Demonstration of Beam and Slab Bridge.

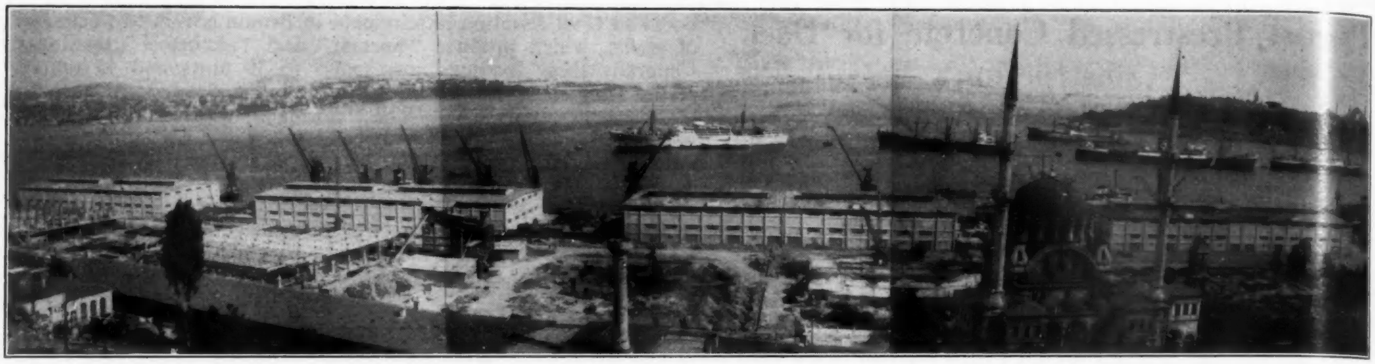
It is somewhat surprising to note that a large number of modern quay designs do not adopt highway bridge loading criteria and tend to adhere to a minimum live loading figure, and it has been suggested that, particularly where in any case heavy crane loadings are catered for, it is worth while to construct the remainder of the decking so that it is capable of taking the occasional loading of a very heavy road vehicle. The use of a pre-stressed concrete deck design based upon the appropriate flat-plate theory, may make this no more expensive. By virtue of the compact, dense, high-quality concrete necessarily used in pre-stressed work, such construction is very suitable for use in quay and jetty work, and factory-made precast units are of particular value under exposed salt water conditions and where pollution occurs. Attention must of course be devoted to the careful placing of any necessary in-situ concrete after the positioning of a precast prestressed deck, and to a satisfactory method of filling the variable width of joints between the bridge beams, which arises from dimensional inaccuracies in casting and positioning. This is particularly important under exposed conditions in maritime works.

New Aluminium Alloy Lighter for Penang Harbour

An all-welded aluminium alloy lighter, believed to be the first of its kind in the world, was launched recently at the Penang Port Commission's dockyard at Bagan Dalam.

The dimensions of the vessel are, length 72-ft., beam 21-ft., and although weighing only 18 tons it has a carrying capacity of 150 tons or 6,000 cu. ft. The Commission decided to use aluminium in the construction in view of the appreciable reduction in operating costs, the high resistance to corrosion of aluminium making painting unnecessary, so that the vessel is expected to remain in service for long periods with a minimum of maintenance. In addition, the low weight and consequent shallow draught of a lighter built of aluminium makes it easier to tow and manoeuvre, thus speeding up port lighter operations.

The vessel was constructed entirely by the Commission's yard from aluminium alloy supplied by the British Aluminium Company, who also assisted in the design. If proved successful it is the intention of the Commission to construct a further 20 of these lighters.



Dock and warehouse construction at Salipazari, Port of Istanbul.

Recent Progress in Harbour Construction in Turkey

By H. RIZA BERKE
(Vice-President, Dept. of Construction of Railways and Harbours, Ministry of Public Works)

During the last few years harbour construction, together with roads and waterworks, has occupied an important place in the Turkish development programme. The Ministry of Public Works has been authorized by the passing of special laws, to proceed with the construction, extension, improvement and equipment of approximately six fully equipped harbours, three refuge harbours, three auxiliary harbours and a large number of jetties of all sizes for a total cost of 800 million Turkish Liras. Some of these works have already been completed or partially completed and the harbours, together with a number of jetties, have been put into service.

Before passing on to figures relating to these works, it is perhaps advisable to give some information regarding the importance of these works in the economic structure of the Turkish Republic.

An examination of the geography of Turkey will lead to an appreciation of the importance of the harbours in the transportation system of the country. Turkey, surrounded on three sides by seas and consisting of two peninsulas, has a coastline whose length is 7,260 kilometres. This coastline, bearing a close geographical resemblance to that of Italy, and approximately to it in extent, makes Turkey, in size, one of the foremost maritime nations, and Maritime Transportation plays as important a role between the coastal towns as it does with other such countries. The fact that the coastal district lying between the Central plateau of the Anatolian peninsula and the coast is extremely rugged and mountainous with few points of access, further stresses the importance of maritime traffic. The coastal towns and harbours have therefore been established in situations where inland routes are possible.

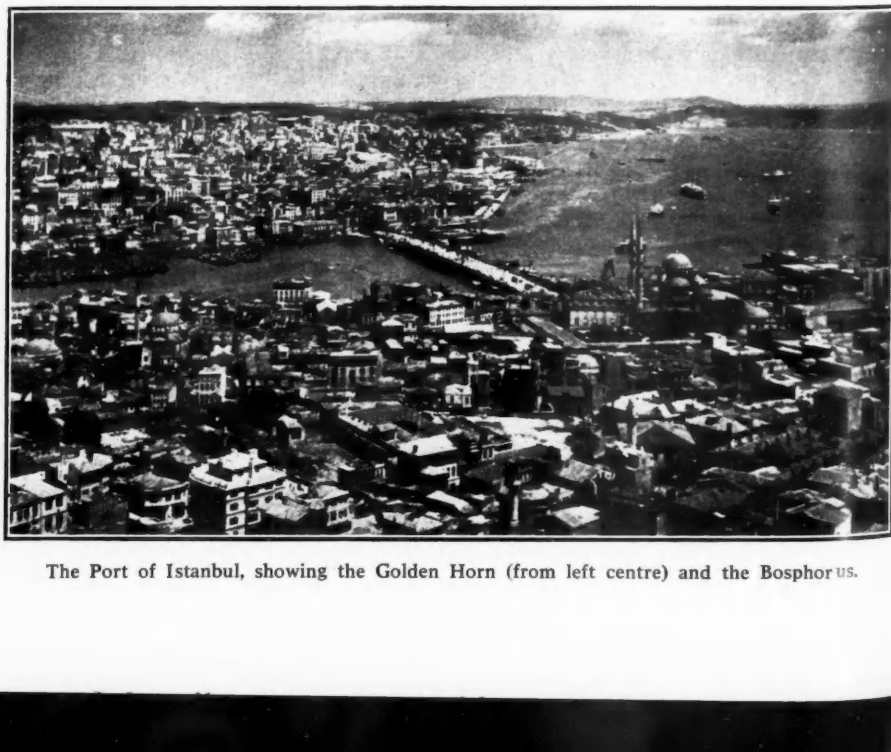
The physical features of the Anatolian plateau and the coastal district have given rise to the formation of natural economic sectors whose activities are centred on coastal towns and harbours, such as Trabzon, Samsun, Iskenderun, Mersin, Izmir, etc. Each of these districts, which have areas of approximately 100,000 square kilometres, are connected to the coastal towns and harbours by roads and railways located along the most economical routes.

During the last few years progress in the control of water-courses, the construction of dams, with a resulting increase in power production, the drainage of swamps and the development of irrigation schemes have greatly contributed towards increased production within each of the harbour areas. Furthermore, the construction of roads complying with international standards has linked the production centres to each other and to the main thoroughfares serving the harbours. In order to cope with the increasing demand, new ships have been bought and put into service and installations capable of dealing with the gathering, distributing and handling of goods are being provided.

Judged by the volume of traffic handled, the harbours have been divided into two groups: Fully equipped and auxiliary harbours and refuge harbours and jetties. Fully equipped harbours are being constructed at Samsun, Istanbul, Izmir, Mersin and Iskenderun. These will include, in addition to commercial docks and facilities for ship repairs, fish docks, oil and industrial cargo handling, free zones where necessary and military bases. The plans and capacities of these harbours are shown on the following drawings and table. Also shown are harbours coming under the second category. These are intended to serve as commercial harbours and have been built at Trabzon, Giresun, Inebolu, Eregli and Kefken.

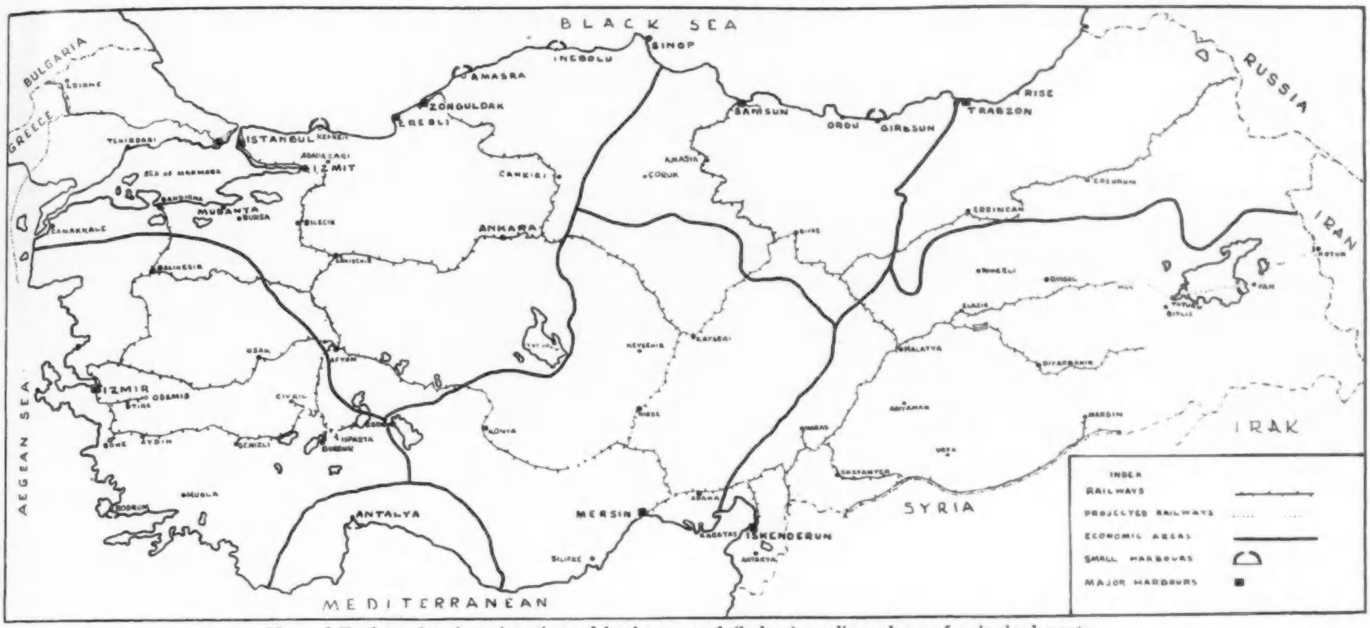
Among future works coming under this category may be quoted, Hopa, Rize, Izmit, Derince, Tekirdag, Bandirma and Edremit, and Antalya will shortly be opened as a fully serviced harbour. In addition to the above, 37 other auxiliary harbours intended to accommodate small craft and fishing vessels have been included in a three years programme which is at present under discussion. Antalya harbour has been included in this programme together with the seven other harbours mentioned above.

The scale on which Turkish harbour construction is proceeding may be seen in the

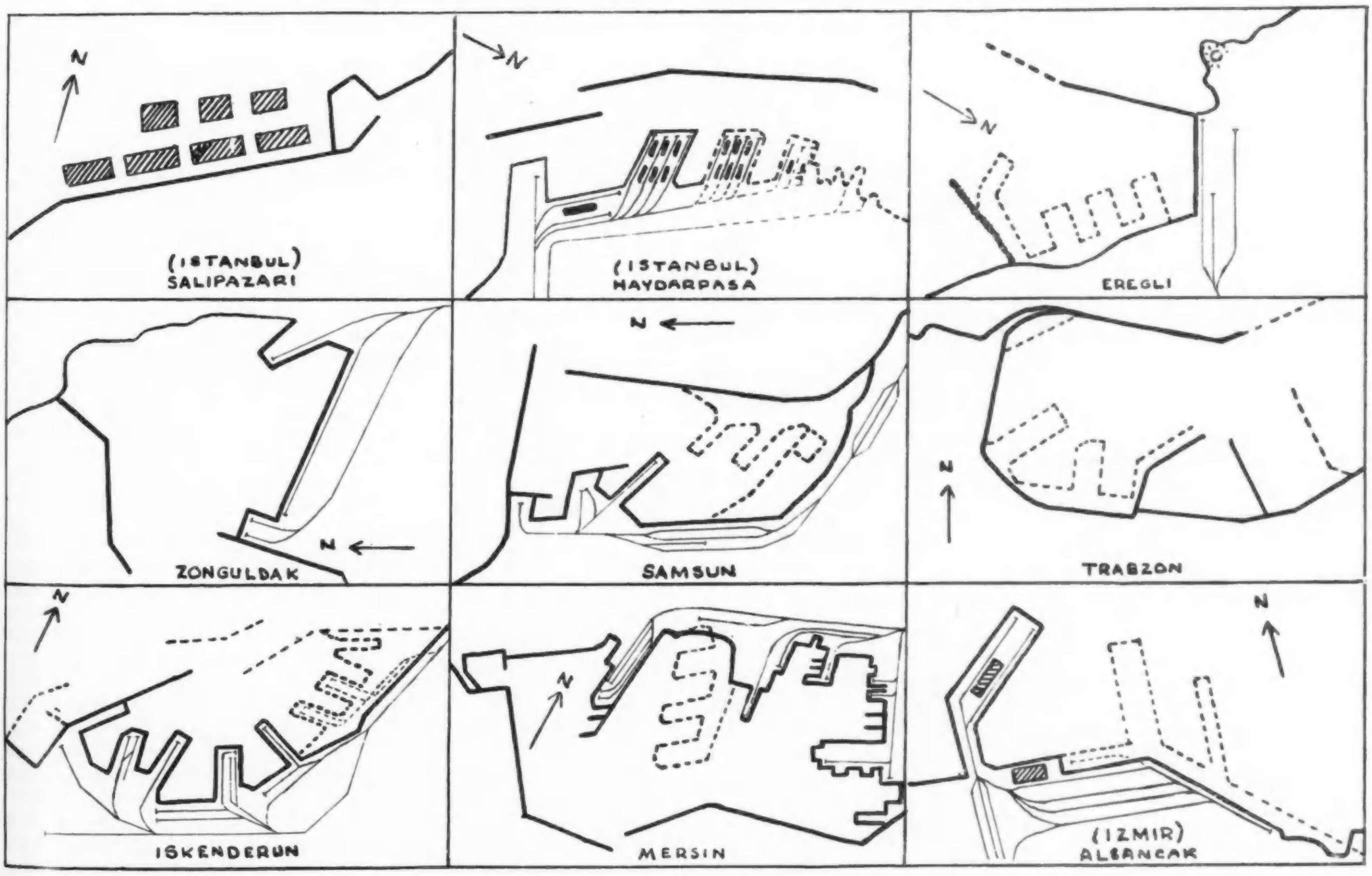


The Port of Istanbul, showing the Golden Horn (from left centre) and the Bosphorus.

Recent Progress in Harbour Construction in Turkey—conitnued



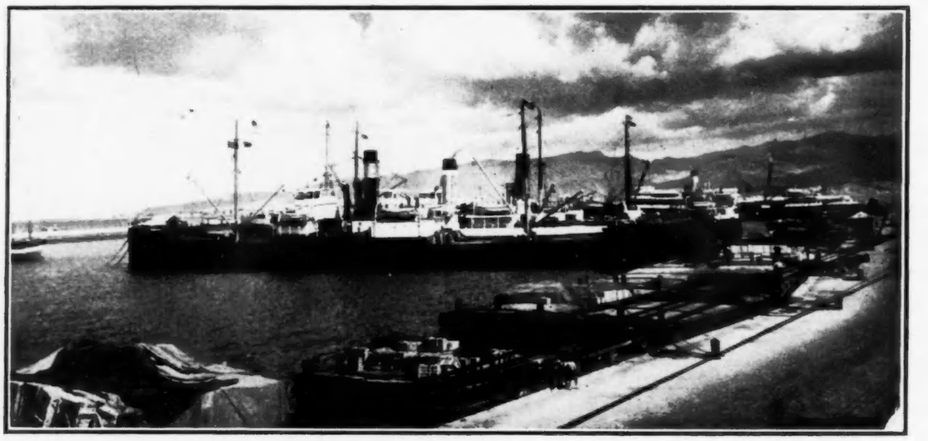
Map of Turkey showing situation of harbours and (below) outline plans of principal ports.



following figures, which relate to works begun after 1950 and either since completed or still in course of construction: Total length of breakwaters 14 kilometres, total length of docks or quays 13.5 kilometres, total area of harbour basins 862 hectares, total area of reclaimed land 408 hectares, total roofed area 160,750 square metres, total grain silo capacity 180,000 tons. To the above may be added the following figures relating to Iskenderun harbour; Length of breakwaters 1.5 kilometres, length of docks 4.7 kilometres, area of reclaimed land 100 hectares, area of harbour basin 62 hectares, roofed area 31,000 square metres. The ore, coal and grain handling plant at Iskenderun has hourly capacities of 250,100 and 300 tons respectively.

All the docks are being fitted with modern goods handling installations, the docks at Istanbul and Haydarpasa having already

Recent Progress in Harbour Construction in Turkey—*continued*



The Port of Izmir showing waterfront closely hemmed in by one of the main thoroughfares of the city.

to a new harbour site has made possible a harbour construction programme which is based on plant requirements and there is no wastage of building equipment.

Loading and Unloading Capacities of Turkish Harbours

Introductory Note: The list of harbours below has been divided into two sections: Fully equipped commercial harbours and refuge harbours. 1950 and 1954 capacity figures are based on actual statistical data and show the increase in volume of traffic attained through a general improvement in the harbour facilities during construction, but, generally, before completion. The 1960 figures are based on the approximate handling capabilities of the harbours on completion of the present construction projects while the 1970-1980 figures represent the normally expected goods traffic after the completion of the planned extensions.

been equipped with 25 modern cranes.

It is necessary to include in a description of the general progress of harbour construction, the building of a large number of jetties having a total length of 5,670 metres and a total area of 45,779 square metres. The building of these auxiliary works have served to encourage the population in their sea-faring pursuits and in the building of small vessels for coastwise service.

An important aspect of Turkish harbour construction is the ownership by the Government of heavy construction plant to the value of 70 million Turkish Liras. Included in this construction plant and equipment are to be found such important units as three large capacity dredgers, six floating cranes of 50-60 tons lifting capacity, approximately 30 tugs, a large number of flat top barges, bottom opening and tipping barges and land plant such as locomotives, heavy duty wagons, excavators, portal cranes, transformers and workshop equipment. By allocating the plant in use at a harbour construction site, to be transferred on completion of the work

Harbour	1950 (tons)	1954 (tons)	1960 After completion of improvement or construction Approx. tons	1970-1980 After future extension Approx. tons
A. Istanbul harbour				
a. Old Harbour ...	2,965,000	2,948,000	3,500,000	4,000,000
b. Salipazaritophane ...				
B. Haydarpasa harbour ...	468,000	721,000	2,000,000	4,500,000
C. Izmir harbour				
a. Town	628,000	795,000	1,000,000	1,000,000
b. Alsancak	200,000	331,000	1,000,000	6,000,000
D. Samsun harbour	189,000	417,000	1,000,000	6,000,000
E. Iskenderun harbour ...	452,000	1,110,000	4,000,000	6,000,000
F. Mersin harbour	152,000	500,000	4,000,000	7,500,000
	5,054,000	6,822,000	16,500,000	35,000,000
1. Trabzon harbour	111,000	230,000	500,000	1,250,000
2. Giresun harbour ...	25,000	50,000	250,000	500,000
3. Inebolu harbour	25,000	50,000	180,000	500,000
4. Zonguldak harbour ...	1,490,000	1,700,000	3,200,000	5,000,000
5. Eregli harbour	209,000	450,000	2,000,000	3,750,000
	1,860,000	2,480,000	6,130,000	11,000,000

Progress in Hydraulics Research

Abstracts from the Report for 1957

The annual report of the Hydraulics Research Board for 1957 was published last month and contains a review of the first ten years' work of the Hydraulics Research Station, D.S.I.R. at Wallingford. Summing up, it is stated that a national station for research into open-channel hydraulics has been established on a sound basis and is now a valuable asset to British civil engineering.

The level of activity at the Hydraulics Research Station during 1957 was greater than in any previous year: all the floor space in the new Main Hall was fully utilised and work was in progress in the new wave basins, flumes and other research facilities. Detailed accounts of most of the Station's projects are given in the Report. Summaries of a few of these are given below.

The Karnafuli River

An investigation of the Karnafuli River and the Port of Chittagong is being carried on on behalf of the Government of Pakistan, the work being financed under the Colombo Plan. With the aid of a model, the Station is studying the problems of the need to stabilise the channel between the port and the confluence of the Karnafuli and Halda rivers and how to establish and maintain adequate depths in all parts of the approaches from the sea.

The Karnafuli River and its tributaries drain an area of 5,600 square miles lying entirely within the tropics and subject to a monsoon rainfall averaging 102 inches per annum. There are, on the average, two major freshets during each monsoon period; they cause the river to rise at least 40-ft. at Rangamati, 71½ miles above the river entrance. There were three such freshets during the 1956 monsoon, the most severe lasting for six days during which the river rose 49-ft. above its dry-season level and the discharge reached a maximum of 263,000 cusecs. Discharges of such magnitude are rare; they are capable of neutralising the highest spring tide and producing a continuous ebb flow at Chittagong. The freshets during 1956 caused marked changes in the river upstream of the jetties and the Director visited Chittagong during February, 1957, to investigate them and the unsatisfactory conditions at the river entrance.

The model is being built to a scale of 1/500 in two stages. The Stage 1 Model, now in operation, includes the Chittagong jetty reach and reproduces 16 miles of the Karnafuli River from the upper limits of the port to a section 4 miles from the river's

Progress in Hydraulics Research—continued

outfall into the Bay of Bengal. During the year, the river channel has been re-moulded, in a washed sand of mean size 0.2 mm., to the conditions obtaining in the dry season of 1950/51. The controlled-weir tide generator has been tested and has been shown to reproduce correctly variations in levels due to tides. Automatic water level recorders, which continuously display tidal levels, have been installed at points corresponding with the Port Commissioners' gauges. The labyrinths, representing the tidal lengths of the Karnafuli and Halda above the port limits, have been extensively modified and comparisons have been made between known prototype tidal levels and rates of propagation of tides and the corresponding information taken from the model under the appropriate scaled-down fresh water flows. The investigation now centres on an attempt to reproduce in the model the channel fluctuations which have occurred in the river upstream of Chittagong during the past seven years, combined with a close study of how these changes affect silting in the jetty reach.

Construction of Stage II of this model will commence as soon as floor space becomes available. In Stage II, the river channel will be extended to the sea and the model will include a stretch of coastal water 5 miles long and 2 miles wide. When these extensions have been completed, the model will be used to study the question of the depths in the approach channels.

The effects of the Kaptai Dam Scheme, at present under construction 40 miles upstream of the port, are of the utmost importance to the investigation. The whole river from Chittagong to Kaptai has now been surveyed and the survey will be repeated periodically after closure of the dam, now scheduled for completion in 1960.

The Lune Estuary

The field studies carried out in the Lune Estuary were briefly reported in last year's Report. The results have now been analysed in a paper entitled "The Long Term Effects of Training Walls, Reclamation and Dredging in Estuaries" which was presented to the Institution of Civil Engineers in April, 1958. In this paper, changes in the Lune Estuary are compared with those which have taken place in the adjoining estuary of the Wyre, in the Thames Estuary and in the Wash.

Before the construction of training works in the Lune Estuary during the period 1847-51, conditions in the estuary were similar to those prevailing in the Wyre. A very heavy charge of suspended material is carried into the estuaries, due to the great range of tide in Morecambe Bay. Much of this silt deposits at high water in the inner estuaries, some of it being removed by the falling tide, but much of it accumulating to form silt banks. Surveys carried out between 1838 and 1845 show that, in the years before training works were constructed, the low water channel between Snatchems and Bazil Point in the middle stretch of the inner estuary had meandered continuously among shifting silt banks. The periodic meandering cut away the accumulated deposits, and so maintained the regime of the estuary, but when this natural erosional process was suppressed due to the channel flow being held permanently along the face of the training wall, the shoals outside the channel silted rapidly. The channel improved in the trained length immediately after the wall was constructed—as was clear from the survey of 1851—but the improvement was short lived due to accretion reducing the cubature and hence the ebb discharge of the estuary. The increase in the area of saltings is clearly shown in Fig. 11 which depicts conditions in 1955-56. A comparison of the surveys of 1838-44 and 1955-56 shows that since training, there has been an accretion of about 35 million cubic yards, reducing the tidal volume at high springs by 47 per cent.; the salting having advanced as shown in the figure.

While these changes were occurring in the inner estuary, quite different changes were occurring to seaward of the training wall. Here, the flood and ebb tides flowed much more directly up and down the estuary and the bed sand moved not by erosion at a cutting face, as in the upper estuary, but in ripples. As the discharge decreased, the total energy of the ebb tide also decreased, whereas the energy of the flood tide, which is generated by the tidal wave, was much less affected. As a result, more sand was carried upstream on the flood tide than was returned downstream on the ebb. For this reason, sand accumulated in the original deep channel and, for a length of 2½ miles below Sunderland Point, the depths of the channel decreased by 8-ft. or more. For a further mile, the loss of depth was 4-ft., and the total accretion of sand in the stretch of the old low water channel seawards of the training wall amounted to 8½ million cubic yards. These results suggest that if a training wall is constructed in an estuary in which the movement of silt is an important factor, salt marshes will tend to build up on either side of the trained channel and shoaling may occur in the main channel to seaward of the works. They also indicate that, if the saltings in the inner estuary were to be reclaimed, the shoaling downstream would be even greater.

Though training works almost inevitably lead to some loss of energy and cubature and hence some overall deterioration, they may nevertheless be justifiable to improve local conditions in the case of large estuaries with a limited supply of material from the sea. It may also be necessary in some cases to construct embankments on both sides of an unstable channel in such an estuary to improve its navigability, even though this would lead to a reduction of cubature and a narrowing of the channel. The Lune investigation has, however, shown that such training walls would not reduce the depths in the trained channel and so may be acceptable provided the training walls extend to deep water, and maintenance dredging is carried out to seaward of the walls, where shoals form due to the sudden expansion of flow on the ebb.

The Thames Estuary

An extensive report on the Thames investigation was presented last year to the Institution of Civil Engineers*. An outstanding point is that all the considerations indicate that dredged material should not be put back into the estuary, but should be pumped ashore. It was announced last year by the late Lord Waverley, then Chairman of the Port of London Authority, that the Authority had decided to follow this recommendation. This change in practice would not be expected to have an immediate effect on the estuary. Gradually, however, the balance would change until eventually a considerable improvement would occur, with a corresponding reduction in the amount of dredging required.

Siltation in Royal Albert Dock Basin

A further problem being investigated by staff of the Port of London Authority, under the direction and with the assistance of the Station, concerns the heavy siltation which takes place in the Albert Dock Basin of the Royal Docks. A large volume of water is pumped from the river into this dock system each day, by way of the Albert Dock Basin and an intake in Gallions Reach, to maintain the impounded water level in the docks. The water so pumped contains an appreciable load of suspended material which settles in the Basin, making it necessary to dredge up to 300,000 hopper tons per annum. Since 1949 many samples have been taken from the pumps, from the intake and its environs, and from various levels in the main channel of the river opposite the intake to determine the source of the silt and the optimum times for pumping in relation to the tide. A model of the intake and part of Gallions Reach has now been built at Victoria Dock to a scale of 1/40 to examine methods of excluding the most heavily silt-laden bed water from the intakes and experiments are in progress.

The Mersey Estuary

A special survey team has been formed for this investigation, which presents problems that may be briefly described as the deterioration in capacity of the upper Estuary, the bars in the

* The Regimen of the Thames Estuary as Affected by Currents, Salinities and River Flow—by Inglis, C. C. and Allen, F. H., Proc. Instn. Civ. Engrs., August, 1957.

Progress in Hydraulics Research—continued

Eastham and Garston channels and the advisability of continuing with the extensions to the training bank on the south side of Queen's Channel. Two models are being used and the construction of the first, which covers the estuary upstream of Rock Light, has been completed. It is equipped with a weir-type tide generator capable of reproducing the natural spring-to-neap tidal cycle. The second and smaller model will be a simulation of the rigid bed of Liverpool Bay.

Coleraine Harbour

A field study of Coleraine Harbour, which is located on the estuary of the River Bann, was undertaken on behalf of the Government of Northern Ireland to investigate shoaling in the harbour, wave action and the formation of a bar between the moles. A survey party from the Station made observations on the estuary in June and July, 1957, and further data are at present being collected. The investigation will be described at length in next year's report.

Eyemouth Harbour

Eyemouth Harbour is on the east coast of Scotland, situated in a rocky bay at the mouth of a small river and is used by some fifty fishing vessels. Under certain wave and tide and conditions, a bar of sand builds up across the entrance to the harbour and has to be removed by dredger or dragline. The Harbour Trustees in association with the Scottish Home Department are considering several proposals for preventing this accumulation of sand and have requested the Station to examine these by means of a model. A 1/40 scale mobile-bed model has been built for the investigation, equipped with a new form of tide-generator and pneumatic wave-makers. The proposals submitted for test included extensions to the piers, sluicing schemes and the construction of an entirely new entrance to the harbour. The testing of these is well advanced and, it is hoped, will be completed early next year, when the work will be reported more fully.

Hong Kong Harbour

For several years, land reclamation schemes within Hong Kong Harbour have been encroaching on the harbour area but the number of these schemes has increased and a complete airstrip is now under construction, projecting some 7,000-ft. into Kowloon Bay. Fears have been expressed that such reclamations, if continued, might have an adverse effect on the tidal streams and consequently, on navigational channels within the harbour. A model study has been undertaken to investigate the effects of these and proposed future reclamations. As the tidal streams pass through the harbour between the island and the mainland, a tide generator will be required at each end of the model. Data are at present being collected and a start has been made on the construction of the model.

Great Yarmouth Harbour

A model investigation of the entrance to Great Yarmouth Harbour was started during the latter part of the year. A scour hole has formed at the entrance to the harbour which, if it were to increase in size or change in position would undermine the piers forming the entrance. Much of the north pier has been reconstructed recently and it is planned to reconstruct the south pier also but, before this is done, conditions at the entrance are to be studied in a model to determine whether it would be possible to eliminate or limit the scour by changes in the configuration of the piers. The model will have a horizontal scale of 1/120 and it is intended to reproduce flows into and out of the harbour, and the tidal streams past the entrance with depths appropriate to the stages of the tide during which violent movement takes place at the harbour entrance.

Ship Mooring Problems arising from Wave Disturbances at a Jetty

A model study has been made of the movement of ships moored at three jetties at a recently developed port in the Commonwealth. The jetties are projected from ocean waves by an island and by the shallow reefs on either side of it. The protection is not complete, however, and from time to time waves reach the jetties disturbing ships moored alongside, so that they damage their plating against the jetties and, occasionally, break their mooring cables. The waves that cause the disturbance appear to be about 2-ft. in height and to have a period of about 20 seconds. The long period of the waves results in horizontal movements of water which are sufficiently sustained to move the ships. Although waves approach from different directions they are refracted so that the alignment of their crests does not vary at the jetties.

Possible solutions to the problem that were to be examined included (a) the construction of wave-reflecting walls in line with the front of the jetties, either short ones along the face of each jetty or a long continuous one connecting the three jetties together; in effect, the ship would then lie at a solid quay instead of at an open jetty; (b) the re-alignment of the jetties to a direction parallel to that of the movement of water; (c) the use of long-travel gravity fenders in the place of the existing rubber fenders.

An undistorted model of the relevant area of the port was built to a scale of 1/60. It was provided with models of the jetties and three models of vessels of different sizes varying between 15,000 and 70,000 tons deadweight (Plate 6). Perfect dynamical similarity can be obtained in a study of this type if the waves, the ship and the jetties are reproduced geometrically, if the densities are the same in the model as in the prototype, and if the elastic modulus of yielding materials, namely the cables and the fenders, is 1/Scale of the elastic modulus of the prototype materials. It was not easy to find materials having the elastic modulus necessary for reproducing steel cables, sisal cables, and rubber fenders and these items were not reproduced geometrically to scale, but their load-deflection characteristics were to scale. All the cables, whether steel or sisal, were reproduced by nylon yarns, some spun specially for the purpose. The rubber fenders were reproduced by metallic springs designed to give the necessary rising spring characteristics of rubber in compression. The forces on the fenders were determined from the extent that they were deflected under impact. In the model tests, the forces were measured under a variety of mooring conditions that included tight ropes, very slack ropes and ropes of intermediate slackness. The effect of a 30-knot wind was reproduced by applying a constant force to the model. The waves reproduced in the model were 2-ft. high and of 20 seconds period. Forces were measured first with the ships at the jetties arranged as at present, and then with several modifications.

The experiments showed that the construction of either type of wave-reflecting wall would not appreciably reduce the impacts, for although it reduced the movement of water-particles in the on-off direction to zero, the motion along the jetties was magnified. A combination of an increased motion of a vessel along the jetty with oblique mooring cables set up a motion of the vessel in an on-off direction and led to impacts on the fenders little different from those under existing conditions. The wave-reflecting wall might have been a solution had the waves approached with their crests parallel to the jetties. There would then have been a true standing wave at the jetty and a vessel there would have had only a vertical motion.

The realignment of the jetties to a direction parallel to that of the movement of the water resulted in the forces on the ships being reduced to one third of their present magnitude; this represented a substantial reduction but not sufficient to ensure that the ships would not be damaged. The third proposal to be tested—the gravity fenders—gave very promising results, however; the forces during impacts were reduced to 1/20 of those recorded with the existing fenders. The tests were made with five fenders, each weighing the equivalent of 15 tons in water, suspended on 6-ft. long chains. Gravity fenders are fenders suspended in such a manner that as they are displaced from their equilibrium position by the impact of the ship, they are also lifted. They do not arrest the movement of the ship suddenly but slow it down until the direction of movement of the water reverses and tends to move the ship away from the jetty once again. The action of the gravity fender is in marked contrast to that of the stiff short travel fender which is compressed in absorbing the kinetic energy possessed by the ship on impact.

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New Cold Store at Southampton Docks

Description of Building and Refrigeration Plant

By H. W. EVANS, A.M.I.C.E.

A COLD STORE has recently been constructed at Southampton Docks replacing one built in 1902 which was destroyed by enemy action in 1940. The store was designed and constructed under the direction of the Docks Engineer, Mr. J. H. Jellett, O.B.E., M.I.C.E. The main contractors employed were Sir Robert McAlpine & Son Ltd., for the building and civil engineering works; Messrs. L. Sterne & Co. Ltd., for the refrigeration plant; and W. A. Taylor Ltd. for the insulation.

This building, which has taken two years to build, is being operated by the International Ice and Cold Storage Co. Ltd., and is registered with Lloyds. It is sited at the New Docks alongside a deep water berth giving facilities for unloading ships of large capacity. Road and rail services are provided for despatching refrigerated cargo. The building is 250-ft. long, 150-ft. wide and 60-ft. high, and has four floors. The ground floor is used for transit purposes and is equipped to deal with road and rail traffic. Four 30 cwt. lifts provide a link between the various floors, and are situate in the middle of the building. The first and second floors are refrigerated, and the third floor is designed for refrigerating at a later date. It is anticipated that when fully equipped the store will hold 7,000 tons of frozen cargo.

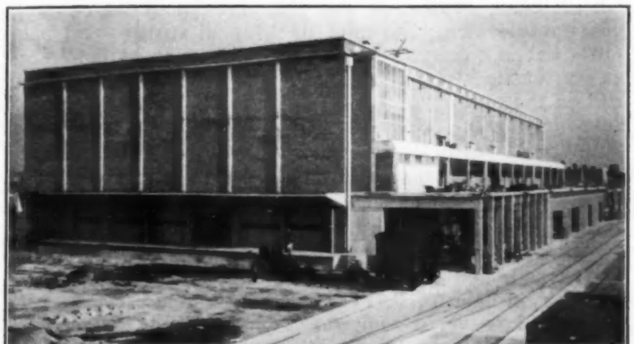


Fig. 1 is a view of the western end of the building, showing the road loading bay covered by a cantilevered concrete canopy. The doors are of the "up and over" pattern, and have observation panels at eye-level for police and customs inspection. The figure also shows the quayside elevation. Cargo can be unloaded from ship direct on to the quay, or on to balconies at first and second floor levels by quayside cranes. Numerous refrigerated doors have been provided on the balconies for rapid transfer of frozen cargo to the cold chambers. The glazed feature in the foreground houses the emergency staircase and also the toilets. Fig. 2 shows the rear elevation, and the junction made with an existing transit shed.

Fig. 3 shows a typical cross-section through the building. The construction is reinforced concrete for all floors and columns, and the external walls are built in 9-in. reinforced brickwork. Cork is used for insulation and is generally in 8-in. thickness. The cork insulation as far as possible envelopes the whole refrigerated space to stop heat leaking into the cold chambers from outside. It has not been possible for structural reasons to provide a complete envelopes of insulation, and links at third floor and roof have been made with the external cladding. At third floor level the cork is fixed to top and underside of the slab. The cork fixed to the underside of the slab is called feathering and is 5-ft. wide. This feathering strip stops warm air penetrating

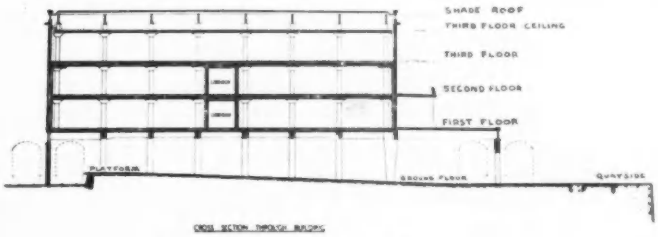
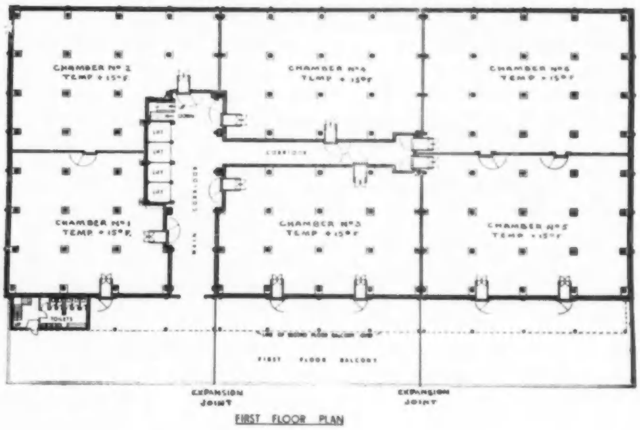


Fig. 1 (top left) Fig. 3 (above)
Fig. 2 (bottom left) Fig. 4 (below)



through the external cladding, through the concrete floor into the chamber. The heat must travel by conduction through 5-ft. of concrete slab to enter the chamber. This 5-ft. of concrete has an insulation value equivalent to 8-in. of cork.

The double roof consists of a R.C. slab, covered by a light asbestos roof placed 6-ft. above the slab. The light roof is supported by steel trusses resting on R.C. stub columns. The external roof cladding is asbestos cavity decking, and roofing felt, which provides protection against the weather, and also minimises the possibility of the top refrigerated floor being affected by the excessive heat in warm weather.

Fig. 4 shows a plan of the first floor, the remaining upper floors following the same pattern. This floor is divided into six chambers, approx. 80-ft. x 60-ft. x 10-ft. high. Corridors provide the links between landing balconies and lifts to the various chambers. The internal partitions are built in slab cork reinforced with a centre layer of steel mesh and rendered to provide a core for stability. Two doors are provided for each chamber to allow flexibility in operating the store.

The chambers on the first floor are designed for a temperature of 15°F. using the direct expansion system of refrigerating. Two

New Cold Store at Southampton—*continued*

chambers in the second floor are designed to operate at −10°F. and −20°F. The remaining four chambers on this floor are designed to operate between 15°F. to 35°F. using air coolers.

Foundations

The structure is supported on approximately 500 16-in. x 16-in. R.C. piles which are generally grouped in fours forming a cap to carry column loads of 240 tons. A typical pile cap is illustrated in Fig. 5. The method of design is conventional, the cap being designed as a truss. The struts are formed by imaginary inclined

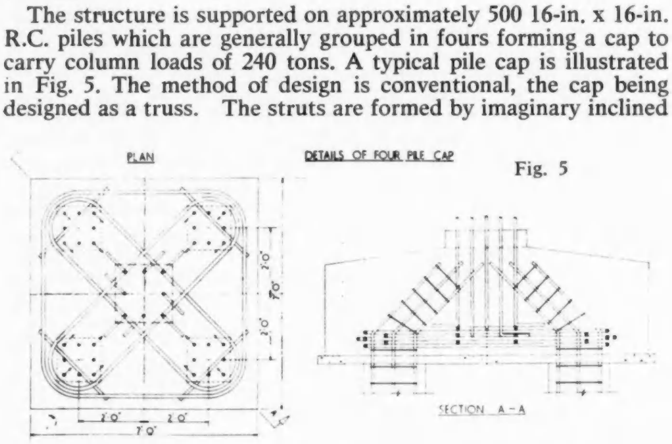


Fig. 5

members running from column to each pile, the reinforcement in each pile being carried up into the cap and inclined to form the reinforcement in the struts. These struts are assumed to be of the same dimensions as the pile, and are treated as short columns. The load carried by the strut is the diagonal force exerted by the column, divided by the number of piles in the group. The horizontal force tending to spread the piles is taken by large loops and links as indicated in the figure.

This method of design is satisfactory and inexpensive, providing the pile group is a compact one, and that the distance between furthest pile and column does not exceed 5-ft. It has been found that the angle of the strut should be 45° to give the best results. The depth of cap can then be kept within reasonable limits, and the horizontal forces can easily be taken by loops or links. If the angle is increased to say 60°, the horizontal forces are reduced, but the depth of cap is increased which is not usually economical. If on the other hand the angle is reduced, the depth of the cap is reduced, but since the horizontal forces increase rapidly, the angle should generally speaking be not less than 40°.

In this cap a 3-in. weak bed of concrete has been placed as a working base. This bed is not essential to the pile cap design, but does allow the work of constructing the cap to be carried out under reasonable conditions, and produces a satisfactory result. A further way of assisting the man on the site is the provision of upstands on top of the cap, in section the same as for the column. This upstand is placed after the cap is complete, and serves to locate the column shuttering accurately.

The shoulders of the cap are sloped, as the concrete in the top corners of the cap serves no useful purpose. It has been found that this angle should not be more than 15°, which is the angle of repose of newly placed concrete.

The loops and links used as ties are made of 1¼-in. mild steel, joined by welding. This method is cheaper than lapping the bars for the necessary bond length, especially with large diameter bars.

Design of Building

The building which is constructed in reinforced concrete in accordance with C.P.114, has four floors which are designed to carry a live load of 2½ cwt. per sq. ft. and a total load of 450 lbs. per sq. ft. The first floor is conventional beam and slab design, the slabs spanning in two directions. The remaining floors are designed as flat slabs in the conventional manner, the main reason for this being to satisfy the requirement of the refrigerating engineer for flat ceilings with the minimum number of projections so that the cooling pipes could be fixed close to the ceilings and give maximum space for storage.

A further requirement is that the whole building should be enveloped with insulation to maintain the low temperature at which the store operates. This means that the internal structure has to be divorced from the external cladding. The sections (Figs. 3 and 8) show how this has been carried out. It was not possible to give a complete insulation envelope, and as mentioned before connections between external and internal structure have been provided at 1st, 3rd and roof levels.

Materials

The design of the columns were based on the use of 1 : 1½ : 3 concrete and that of the foundations, beams and slabs on a 1 : 2 : 4 mix. In actual fact the mixes were made slightly richer to obtain strength and workability for the concrete. All concrete in the superstructure was vibrated, and advantage was taken of the allowable increase of compressive stress of 10% as permitted in C.P.114. Lined shutters were used throughout, being made with ⅝-in. plywood faced with 1/16-in. plastic. The finished surface of the concrete proved to be excellent, as shown in Fig. 6, and it was found that the shutters could be used about 12 times.

In the main, Tentor bar was used throughout the building for main reinforcement, and mild steel for all stirrups and binders. Tentor bar is a deformed mild steel bar which by cold working has its tensile strength raised by approximately 50%, and in addition the grooves in the bar increase the bond strength. The allowable working stresses for this reinforcement were taken as follows:—

Bars in tension (other than shear reinforcement) ...	27,000 lb. per sq. in.
Bars in compression	20,000 lb. per sq. in.
Bars in tension due to shear	20,000 lb. per sq. in.
Local bond stress 1:2:4 mix	225 lb. per sq. in.
Average bond stress 1:2:4 mix	150 lb. per sq. in.

The tensile and compressive strengths taken compare favourably with the latest C.P.114 (1957). The bond stress allowed in the design however is somewhat higher, and this was felt to be justified as a number of authorities have carried out experiments with Tentor and found it to have excellent bond properties. The experiments have shown that the bond of ordinary mild steel round bars fails at the first slip, i.e., approximately 400-lb. per square inch. In the case of Tentor bar, however, the first slip

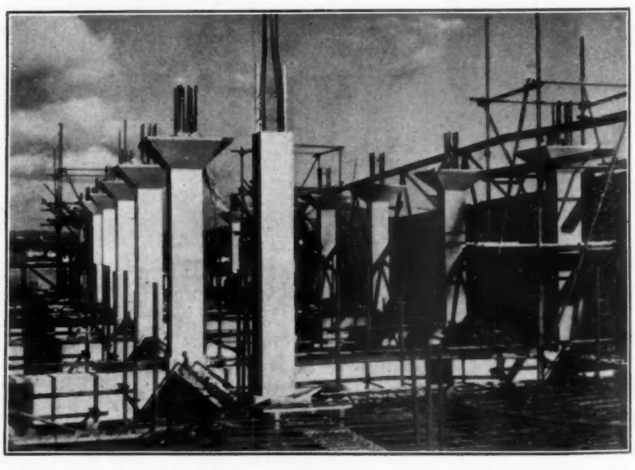


Fig. 6

takes place at about the same figure, but unlike the round bar which fails completely in bond at the first slip, the ridges in the Tentor bite the concrete and anchor the bar. The ultimate failure did in some instances take place at a bond stress of 900 lb. per sq. in., but in others it was not possible to measure the ultimate bond stress as the bars often failed in tension owing to the magnitude of the pull exerted.

Flat Slab Design v. Beam and Slab

The building gives an excellent opportunity to compare the costs of " Flat Slab " and " Beam and Slab " design.

New Cold Store at Southampton—continued

The spans and loads were identical for both types of construction, the size of panels being 20-ft. 5-in. x 15-ft. 7-in., carrying a total load of 450 lb. per sq. foot. The depth of slab in the flat slab design was 9-in. and in beam and slab 7-in. with the corner panels 8-in. deep. The amount of Tentor steel required per sq. yard was:

33.6 lb. per sq. yard for Flat Slab
and 60.3 lb. per sq. yard for Beam and Slab design.

Note that for mild steel these figures should be increased by one-third.

The cost for Beam and Slab per sq. yard was £5.

The cost for Flat Slab per sq. yard was £3 14s., which is a saving of £2 per sq. yd. with Flat Slab Design.

A further point in the favour of Flat Slab Design is that in multi-storey construction approx. 12-in. is saved in height for each storey as there are no obstructions such as beams to restrict the headroom.

Striking Formwork

On a large R.C. building the question of striking formwork is always an important problem, both to the engineer and to the contractor, who is generally anxious to strike the shuttering as soon as possible, so that it can be used elsewhere. The engineer is anxious that the work should proceed rapidly and efficiently, and is faced with the problem of ascertaining the actual strength of the concrete to find out when it will be self-supporting. This strength factor is largely governed by the temperature during the particular period of concreting. A standard way of doing this is by laying down in the Specification the minimum intervals of time allowed to elapse between placing of concrete and the striking of shuttering for the different cases, as follows:—

	Cold Weather Just above freezing Days	Normal Weather about 60 deg. F Days
Beam sides, walls and columns ...	6	2
Slabs (props left under)	10	3
Beam soffits (props left under) ...	14	7
Removal of props to slabs ...	21	7
Removal of props to beams ...	28	16

These figures are given for ordinary Portland Cement and some reduction can be made for Rapid Hardening Cement. If a frost should occur the removal of the formwork is further delayed for the duration of the frost. This last point is always a bone of contention between the contractor and the resident engineer as the duration of a frost is arbitrary unless a thermograph is kept on the job giving a continuous record of temperature by day and night.

In view of the large amount of concrete being placed in this building, a special arrangement was made with the contractor, which proved to be satisfactory to all parties. The engineer prepared a table showing the minimum concrete strengths required for the removal of the shuttering and props to newly laid concrete. It was not necessary to allow very high concrete compressive stresses as the floors and beams had only to carry themselves and a light live load. The table used was as follows:—

	Minimum compressive stress in concrete
Beams sides, walls and columns	1,500 lb. per sq. in.
Slabs (props left under)	1,500 lb. per sq. in.
Beam soffits (props left under)	2,000 lb. per sq. in.
Removal of props to slabs	2,000 lb. per sq. in.
Removal of props to slabs	3,000 lb. per sq. in.
Loading of all concrete members with superload, i.e., construction of floors above	3,000 lb. per sq. in.

The concrete compressive strengths were based on the average of 3 cubes taken from the concrete actually deposited in the units under consideration. These cubes were placed alongside the beams or slabs on the site and cured under the same conditions. At a suitable time after the placing of concrete one cube was cracked to find its compressive strength. If satisfactory the remainder were tested and the average result taken. If however the first cube did not give the required strength, the tests on the remaining cubes were delayed until it was thought that the compressive strength would be reached. This method proved quite successful as the R.E. knew that when the shuttering or props were removed the concrete was strong enough, and the contractor found that the shuttering and props could be removed a few days earlier than originally allowed.

Insulation of Cold Store

The Cold Store at present has two floors insulated. The third floor will possibly be insulated in the future. There are six chambers on each floor; those of the 1st floor were designed to hold temperatures of 15°F., the two chambers on the second floor to hold temperatures of −10°F. and −20°F., and the remaining four chambers to hold temperatures from 15°F. to 45°F.

When designing the insulation for this type of building, the designer has to decide the optimum amount of insulation required from the financial point of view, which will be economical in operation when taking into account the capital cost, depreciation and amortisation, maintenance of plant and insulation, and the operating cost of the refrigerating plant.

The basic laws for insulation design are:—

(a) Heat flows from a warm body or space to a colder one, and
(b) Water vapour will endeavour to flow from a region of higher vapour pressure to a region of lower vapour pressure.

Dealing with each law of nature in turn,

(a) It is not possible to prevent completely the flow of heat between two regions at different temperatures, but the correct use of an adequate amount of suitable insulation will reduce the heat flow sufficiently to enable low temperatures to be conveniently maintained by refrigerated equipment.
Heat flow takes place in three ways.
  (i) Direct conduction of heat through any material.
  (ii) Radiation from one surface to another not in direct contact.
  (iii) Convection currents in gases or liquids in direct contact with surfaces of differing temperatures, or subjected to localised temperature changes.
When considering the insulating properties of any particular material, it is necessary to consider the thermal conductance of the finished structure as a whole, that is, all the materials which go to make up the walls and floors.

(b) Water vapour pressure is affected by temperature; the lower the temperature the lower the vapour pressure; other factors being equal, there is, therefore, always a pressure exerted by the vapour, driving it towards the low temperature.
If any vapour penetrates into the insulation, it may at some point during its passage condense into water, saturating the insulation and thereby causing a rise in thermal conductivity. This saturation may also cause the material to swell, damaging the insulation structure, and providing a suitable condition for the growth of " dry rot " should any timber be present.

Further, if this water is converted to ice the increase in volume which takes place during this physical change will damage the structure of the insulation causing cracks and passages which will admit more vapour and create more damage. Ice formation in this manner is a progressive form of destruction.

Fig. 7 shows a typical section through an external wall, and the manner in which the insulation is carried out to overcome the problems mentioned. This section has been taken through the wall insulating the chamber to operate at −20°F. The construction consists of 9-in. brick external skin plastered on its inside with ⅜-in. cement rendering. This rendering is provided to give a smooth surface to apply a vapour barrier which consists of two brushed on coats of Flintkote. This barrier is most important as its function is to stop water vapour penetrating the insulation. Flintkote is a bituminous compound which is extremely good for this type of work.

Three layers of cork built up to a 10-in. thickness have been used as the insulating material. The cork is supplied in slabs 3-ft. x 1-ft. x various thicknesses in inches. The first layer is dipped in hot bitumen and stuck on to the vapour barrier. The method of dipping is interesting, only one base, one side, and the bottom being dipped, so that as successive slabs are placed there is only one thin bituminous layer joining one slab to the next. This procedure is important as it ensures that the bitumen sticking the slabs together is as thin as possible. A thick

New Cold Store at Southampton—continued

bituminous joint is detrimental as it represents a weak point in the insulation.

The next layer of cork is laid again in bitumen as previously described and further secured by 6-in. hardwood skewers to the first layer. Care is taken to break the joints as shown in the cross section (Fig. 7). The third layer is fixed in a similar manner, and the internal surface then rendered to protect the cork from damage when handling the merchandise.

This method of construction is quite orthodox and has been proved to be satisfactory in many cold stores. The warm outside air will penetrate the insulation, but the temperature gradient in the cork will fall very sharply; if the outside temperature happened to be 60°F. then the range of temperature would be 112°F. The inside temperature of the brickwork might be 57°F., and the remaining drop in temperature is in the cork. This means that any water penetrating the cork will freeze at about its centre, which would have serious consequences. However the vapour barrier does provide an adequate protection against water vapour and is most important.

Fig. 8 also shows the refrigerating pipes, which are hung from the ceiling; the third floor is as it will be when completed, with a cold air circulation system. Each chamber is provided with two large ducts which are connected to an air cooler, the cold air being regulated by adjustable apertures in the side of the air ducts.

The design of the entrances to the cold chambers is interesting as these are points where the cold losses may be great. Fig. 9 shows a typical entrance in plan and elevation. The door opening is 4-ft. 8-in. wide and 6-ft. 6-in. high. The superfreezer door is approximately 10-in. thick, constructed in timber and insulated with cork, and clad with galvanised sheet metal. The door weighs about 8 cwts. and is provided with special hinges and fastenings so that it operates easily. It is a standard requirement that all doors can be operated from both sides, and a special safety bar is placed on the inside to enable the door to be opened from this side. A further feature is the provision of a heating element fixing on the meeting faces of door and reveal. This element is regulated to give a fixed temperature and is provided

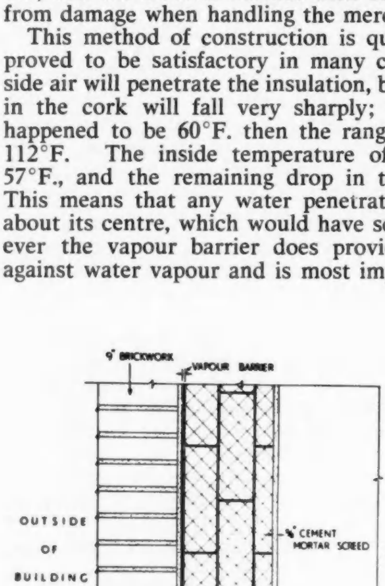


Fig. 7

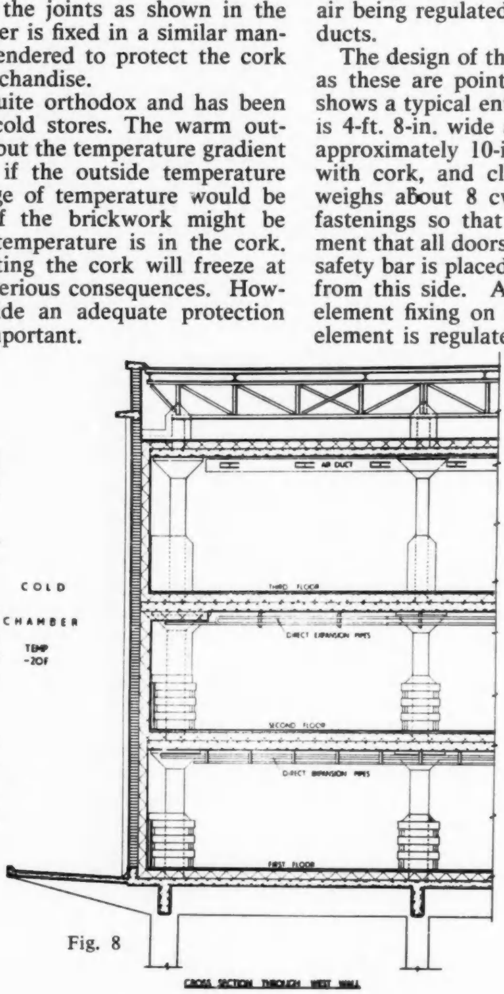


Fig. 8

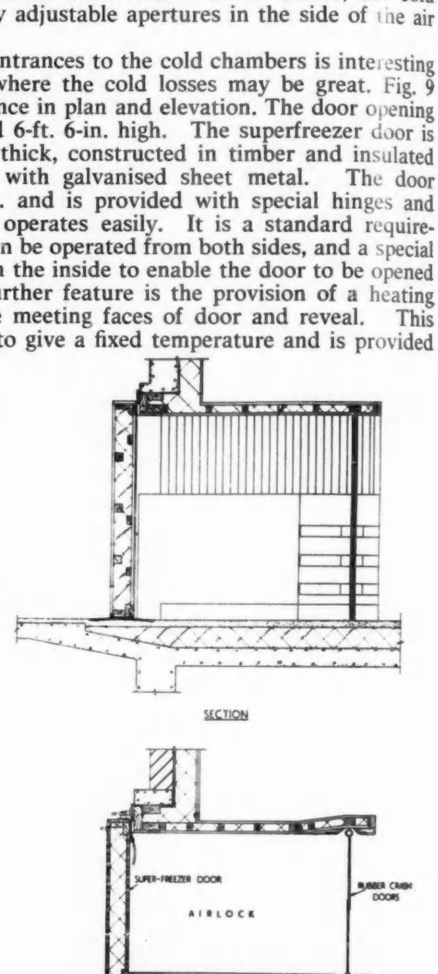


Fig. 9

Fig. 8 is a section through the western end of the building, and shows the general arrangement of the cold chambers. The projection on the left of the section is a concrete canopy over the road loading balcony. The 9-in. thick brickwork is reinforced to give it additional strength. The first floor is insulated with 8-in. thick cork to walls and floors, and the second floor 10-in. cork on walls and 8-in. cork on floors. The third floor shows the future insulation which is similar to the first floor. The floor cork is laid in a similar manner as described for wall cork, i.e., the vapour barrier is laid, on the concrete floor slab, followed by two layers of cork laid in bitumen and secured by skewers. The cork floor is protected by a 2½-in. thickness of granolithic paving reinforced with a light mesh.

The walls are protected by 2-in. x 2-in. vertical timber battens arranged about 12-in. apart and secured at ceiling and floor by 4-in. x 2-in. bearers. Across the face of the 2-in. x 2-in. battens are arranged three 7-in. x 1¼-in. bumper rails. All the timber protection has a dual function; it protects the insulation from damage by carelessly handled cargo, and also the arrangement of timber is such that air can circulate up the walls even when cargo is stacked close to them.

The columns are also insulated with cork to stop the heat being conducted through the concrete into the chamber. This insulation is 6-in. thick and extends 4-ft. up the column. The cork is laid as previously mentioned, and is protected with rendering and timber battens.

to stop ice forming at the meeting faces, which would cause difficulty in opening the door. Each door is also provided with an airlock constructed in timber and insulated with cork. At the end of the airlock, swing rubber crash doors are provided, which are self-closing. This airlock helps to reduce the cold losses when the superfreezer door is open.

Another method used at entrances to cold chambers is the Miniveil. Fig. 10 shows a section and an elevation of a doorway provided with this apparatus. A small 1/9 h.p. fan connected to a bell shaped duct is fixed over the top of the door and covering its whole width. The fan operates when the door is open by a contact plunger fitted to the door reveal and blows a curtain of warm air over the door entrance at an angle of approximately 15° with the vertical. The velocity can be adjusted to suit any requirements but under normal circumstances it is about 16-ft. per second. It is most effective in stopping the transfer of cold air out and warm air into the chamber. The air in the cold store being denser than the outside air causes an increased pressure at floor level as compared with the outside air, and as the door is opened it flows out of the entrance at about floor level. This displacement of air in the chamber allows warm air to enter at the top. Fig. 10 shows

New Cold Store at Southampton—*continued*

how a curtain of air provided by the Miniveil stops this process.

Care must be taken however not to obstruct the veil of air, as for example, when the door is partly open, or a truck is placed in the entrance, as the veil would then be ineffective, and air would be deflected into the cold chamber.

Mechanical Refrigerating Plant

The machinery to maintain the low temperatures required in the cold store is housed in an engine room on the ground floor, and is connected to pipe circuits and air coolers in the cold chambers.

Mechanical refrigeration is one of the many applications of heat energy, and is the reverse of the cycle used for prime movers. In mechanical refrigeration, heat is absorbed at a low temperature and rejected at a higher temperature, a compressor being used to pump a refrigerant around a circuit in which it completes a cycle of operations.

There are many known refrigerants, such as carbon dioxide, Freon 12, methyl-chloride, propane, sulphur dioxide and ammonia. In industrial refrigerating plants ammonia is the refrigerant in common use, and is superior to other refrigerants in

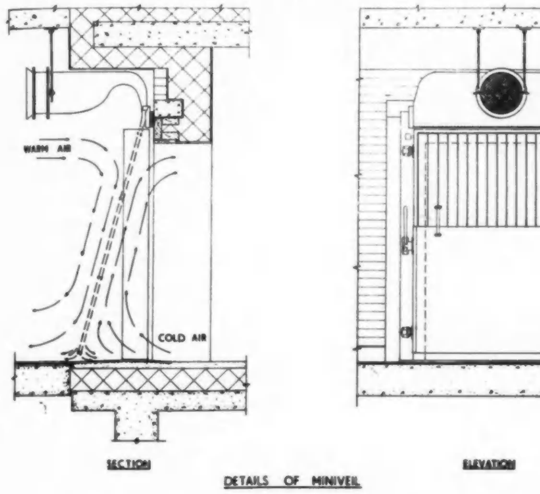


Fig. 10

respect of the amount of refrigeration produced for a given input h.p. The boiling point of ammonia is $-28.0°$F. at atmospheric pressure and it freezes at $-108.0°$F.

Ammonia is used in the new refrigeration plant, which has four fundamental parts—compressor, condenser, expansion valve, and evaporating pipes. The ammonia gas is drawn at a controlled pressure, from the evaporating coils in the cold rooms into the compressor where the gas is compressed to about 170 lb. per square inch and the temperature as the gas rises to, say, 250°F. forming a superheated vapour. This hot gas is then converted to a liquid in the condenser. The condensers consist basically of nests of small bore pipes connected in layers, and surrounded by a sheet metal case. The kind used is the evaporative type where water is continually cascaded over the condensing surface, the condensing being done by the evaporation of the water. Fans are used to circulate the air over the condensing surface. The saturated air is rejected by means of ducts to the atmosphere. In this manner, the hot gas is converted into a liquid, still at high pressure. Thus the condenser changes the superheated gas to liquid, discharging the heat absorbed in the evaporating coils, plus the heat equivalent of the mechanical power required to compress the gas, to the condensing medium, i.e. water or air. On leaving the condenser the ammonia liquid at ambient temperature and high pressure passes through the circuit to the expansion valve. At this point the liquid is allowed to expand at a controlled rate, with a consequent drop in pressure and temperature. The pressure could be 40 lb. per sq. in. and the temperature $-10°$F. The expansion valve which controls the flow of refrigerant at the desired evaporating pressure is connected directly to the evaporating coils, which are suspended from the ceilings of the cold chambers, and in these pipe circuits the liquid enters at low temperature and absorbs heat from the air in the cold chamber. At some point in the pipe circuits the evaporation of the liquid is complete, so that gas only returns to the compressor to complete the cycle.

In all refrigerating plants, the higher the evaporating temperature and the lower the condensing temperature the more efficient is the plant as regards the heat extracted per unit of horsepower.

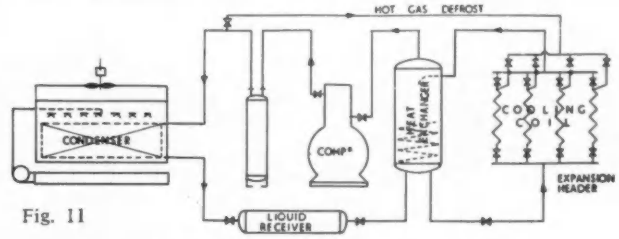


Fig. 11

At Southampton, there are three types of coolers:

(1) Direct expansion, with a heat exchanger to take care of any surplus liquid returning to the compressor.

(2) The forced air circulation batteries on direct expansion designed for flooded operation.

(3) Direct expansion on pump circulation where the low temperature liquid refrigerant is circulated by means of a pump, an accumulator being incorporated in the system.

(1) The direct expansion system is used in rooms 1, 2, 3, 4, 5 and 6 which are held at 15°F. and cooled by means of $1\frac{1}{2}$-in. bore grids affixed to the ceiling of the insulated cold store.

Fig. 11 shows a typical arrangement of a direct expansion system, operating on the cycle earlier described with one or two extra refinements. The fundamental parts are, the compressor, condenser, expansion header and the evaporating or cooling coils. A heat exchanger has been placed between evaporating coils and the compressor, to ensure that only gas is allowed in the compressor, any liquid being collected in heat exchanger. Also a liquid receiver is placed between condenser and expansion header. A further important part is the hot gas defrost. A connection from a point between compressor and condenser enables hot gas to be circulated through the evaporating coils. This is useful for defrosting the ceiling coils, as after a time the coils become coated with thick snow and lose their efficiency as cold radiators.

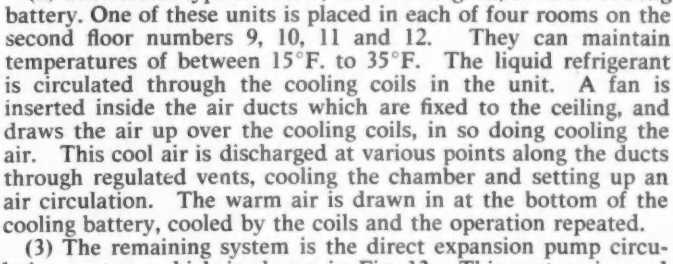


Fig. 12

(2) The second type of cooler, shown in Fig. 12, is an air cooling battery. One of these units is placed in each of four rooms on the second floor numbers 9, 10, 11 and 12. They can maintain temperatures of between 15°F. to 35°F. The liquid refrigerant is circulated through the cooling coils in the unit. A fan is inserted inside the air ducts which are fixed to the ceiling, and draws the air up over the cooling coils, in so doing cooling the air. This cool air is discharged at various points along the ducts through regulated vents, cooling the chamber and setting up an air circulation. The warm air is drawn in at the bottom of the cooling battery, cooled by the coils and the operation repeated.

(3) The remaining system is the direct expansion pump circulation system, which is shown in Fig. 13. This system is used in the low and ultra-low temperature rooms 7 and 8 on the second floor. These can be held at $-10°$F. and $-20°$F. respectively. The $1\frac{1}{2}$-in. grids are fixed to the ceiling and walls and are double-banked to give the necessary cooling surface. The

New Cold Store at Southampton—continued

15°F. to 35°F. rooms are held by single stage compression but for the low temperature rooms the compression ratio would be too great, so a compound or two-stage compressor is used. It is generally accepted that the maximum compression ratio for efficient operation is 7 : 1.

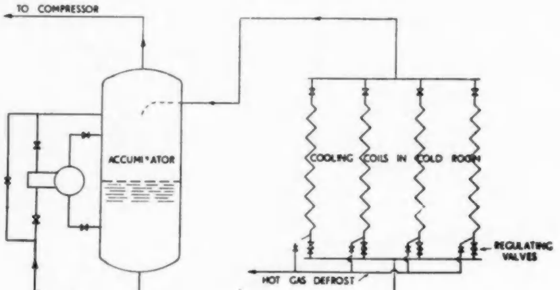


Fig. 13

In this system the refrigerant is expanded at the controlled temperature into an accumulator. The liquid level is controlled by a low pressure type float valve. On passing through the expansion valve, part of the liquid evaporates and cools the remainder of the liquid to the evaporating temperature. The gas which has been formed returns direct to the compressor as will be seen in the diagram. The amount of liquid which flashes is approximately 17% of the liquid supplied from the condenser. The liquid at the evaporating temperature is then pumped to the cooling coils. The surplus liquid which is usually about three times the liquid which can be evaporated in the coils returns to the accumulator with the gas formed from the evaporation in the coils. This surplus liquid drops to the bottom of the accumulator and the dry gas is collected by the compressor.

In the direct expansion system the liquid and gas from the expansion valve pass through the cooling coils. The globules of liquid bounce on the surface of the tubes and are carried by the gas which is formed and drawn by the compressor. The amount of liquid fed to the coils is just sufficient to ensure that all the liquid has been evaporated before it reaches the end of the cooling circuit.

The behaviour of the refrigerant can be likened to the action of globules of water being dropped on to a hot plate. The portion of the liquid which touches the hot plate is immediately evaporated and in changing its state greatly increases in volume. The minor explosion shoots the globules into the air and the drop of water keeps on bouncing until all the liquid has been evaporated.

In the flooded system as used for the air cooling batteries, the behaviour of the refrigerant is similar to the action of water being boiled in a kettle.

The three methods of cooling may seem unnecessary, but these are directly related to the working temperatures.

Methods 1 and 3 are called direct expansion because the refrigerant takes the heat directly from the air in the chamber. The pump is introduced in the system 3 to ensure that the evaporating coils are full of liquid which enables the chambers to be operated at much lower temperatures than with the other method. In method 2 the air is removed from the chamber for cooling. Both systems are suitable when operating a chamber at 32°F. or below, but above this temperature the evaporating coils will drip, and damage the stored foodstuffs. The cold air circulation method covers the range of temperature above freezing.

It will be readily appreciated that to equip a store with only one system would limit the scope of the store, so the three systems are available to give the maximum range possible.

Operation of Store

The first law of cold storage is to keep the commodities at the same temperature and humidity as when it is first frozen. Any fluctuation tends to cause deterioration. This requirement was in mind when the cold store was sited on the quayside, designed with unloading balconies so as to be able to take frozen goods quickly from refrigerated ship to the store, keeping the possible temperature change to a minimum.

The main commodities to be stored can be put under six headings: meat, dairy produce (butter, cheese, etc.), fish, fruit, vegetables, and quick frozen foods, which is a specialised industry. All have what is known as a critical temperature. This is governed by the water content, and whereas some foods can be frozen, others have to be chilled. Table I shows the various products with the storage temperature, the critical temperature, and the storage periods. The margin between the storage temperature and the critical temperature is very small, and so care must be taken to control the room temperature accurately.

TABLE I		STORAGE TEMP °F.	CRITICAL TEMP °F.	STORAGE PERIOD
MEAT	Chilled	30°F	28°F	10-15 Days
	Frozen	15°F	10°F	1 - 10 Months
DAIRY PRODUCE	Butter	15°F	5°F	1 - 6 Months
	Frozen Eggs	5°F	0°F	1 - 2 Years
	Shell Eggs	31°F	30°F	6-10 Months
	Cheese	40°F	35°F	1 - 6 Months
FISH		0°F	5°F	2 - 3 Months
FRUIT	Apples	36°F	31°F	1 - 6 Months
	Pears	32°F	30°F	1 - 4 Months
	Oranges	32°F	31°F	1 - 4 Months
VEGETABLES	Green	35°F	32°F	10-20 Days
	Root	38°F	34°F	1 - 3 Months

COLD STORAGE TEMPERATURES FOR VARIOUS FOODSTUFF

Care must be taken when stowing the goods. At all times there must be a good air circulation around every part of the stow. Anything that is above 32°F. on intake which requires freezing should be laid out singly and then stored when completely frozen. If this not not done then the stow will freeze and still retain its heat inside, which has at times been known to cause self-combustion. Where goods are to be stored at a temperature above 32°F. the humidity has to be considered.

It will be seen that the operation of a cold store is a very specialised industry, where correct temperature and humidity are of the greatest importance to ensure that the thousands of tons of food can be kept fresh and of good quality until required.

Retirement

Mr. E. S. Ely, M.I.Mech.E., M.I.E.E., Mechanical and Electrical Engineer at Southampton Docks retired last month.

A native of Romsey, Mr. Ely was apprenticed with the electrical engineers' department at the docks, and served with the Hampshire Regiment and Royal Engineers in the First World War. After the war he gained experience with Allen West and Company, Brighton, and the English Electric Company, London and Stafford, and returned to Southampton in 1928 as engineering draughtsman. In this position he was associated from the beginning with the development of the new docks in regard to pumping and sub-stations, cranes and installations. Electricity consumption in the port rose from 8 mn. units in 1942 to 33 mn. units last year. He also dealt with all electrical equipment for the two new terminals and carried through a scheme for improved docks lighting.

He was made assistant electrical engineer in 1940, and electrical engineer two years later, taking an active part in the development of schemes for the improvement of mechanical and electrical equipment in the cross-Channel steamers.

Mr. Ely is a member of the Institution of Mechanical Engineers and the Institution of Electrical Engineers. He represented the BTC on crane and crane auxiliary committees of the British Standards Institute, and was chairman of the BTC (Docks) mechanical and electrical engineers' committee.

Mr. Ely plans to continue his professional work in a private capacity from his new home in Sussex.

International Pallet Standardisation

The Fifth Meeting of the I.S.O. Technical Committee

(Specially Contributed)

The above-mentioned matter is being dealt with by Technical Committee 51 of the International Organisation for Standardisation, which first met in December, 1952. At the Committee's fifth meeting, which was held recently in Harrogate, England, the main technical items on the agenda were (1) a glossary of terms relating to pallets; (2) details of large pallets, specially for sea transport and (3) testing of pallets. The constitution of the committee at the time of the meeting was:—

Participating members

Australia, Austria, Belgium, Denmark, Finland, France, Germany, Netherlands, Norway, Poland, Switzerland, Sweden, Czechoslovakia, United Kingdom and U.S.S.R.

Observer members

Present at Harrogate

India, Ireland, Israel, Italy, Japan, Yugoslavia.

Not Present at Harrogate

Bulgaria, Chile, Greece, Hungary, Mexico, New Zealand, Pakistan, Portugal, Rumania, Turkey, Union of South Africa, U.S.A.

Also in attendance to take part in discussions, but without the right to vote, of course, were representatives of the International Union of Railways, the International Chamber of Shipping, the European Packaging Federation, the International Cargo Handling Co-ordination Association and the International Road Transport Union.

Glossary of terms

The meeting agreed to recommend a glossary produced by a Working Group in English, French and Russian, the three official I.S.O. languages. This glossary defines and illustrates 22 terms relating to pallets (such as entry member, bearer, stringer and chamfer) and will be of great help, not only in international committee work but also in the general business connected with pallets.

Large pallets (48-in. × 64-in. and 48-in. × 72-in.)

These are often referred to as "Stevedore" pallets or "Maritime pallets." "Stevedore" pallets are not through-transit pallets, however, but are dock tools; the term "maritime" is vague. I.S.O. therefore refers simply to "large" pallets.

Although the main need for large pallets is to give an economical load for modern quay cranes and ships' purchases, those which are the concern of I.S.O. are, of course, through-transit pallets and must therefore also be suitable for stowing in road and rail transport vehicles by fork lift and pallet trucks. Thus they have approximately the same characteristics as small standard sizes.

The international standard

The recommendations made at Harrogate in connection with large pallets and which it can be expected will in due course be accepted as I.S.O. standards have been incorporated into the following summary:—

Work Completed to Date

(1) Nominal plan sizes of pallets.

Flat Pallets

Small pallets:
- 32 × 48-in. (800 × 1,200 mm.)
- 40 × 48-in. (1,000 × 1,200 mm.)
- 32 × 40-in. (800 × 1,000 mm.)

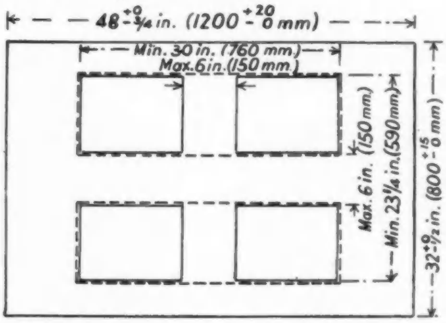
Large pallets:
- 48 × 64-in. (1,200 × 1,600 mm.)
- 48 × 72-in. (1,200 × 1,800 mm.)

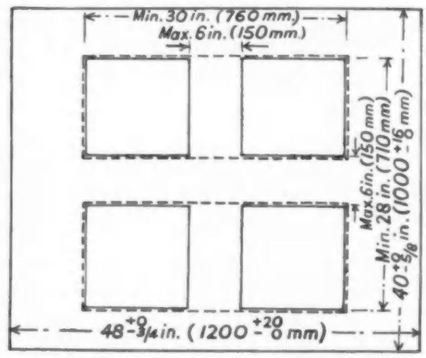
Box Pallets

*32 × 48-in. (800 × 1,200 mm.)
*40 × 48-in. (1,000 × 1200 mm.)

*see paragraph 11

(2) Tolerances to apply. (These tolerances to be regarded as extreme limits, within which each member body will fix its own manufacturing tolerances).





Minimum openings for pallet truck wheels in bottom deck of 2-way and 4-way pallets. If the construction will permit, it is strongly recommended that (i) the minimum openings provided should be those indicated by the dotted lines and (ii) the minimum dimension of 30-in. (760 mm.) shall be increased to 32-in. (800 mm.).

Effective Plan Dimension	*Maximum*	*Minimum*
48 in.	+ 0	— $\frac{3}{4}$ in.
1,200 mm.	+ 20	— 0
40 in.	+ 0	— $\frac{5}{8}$ in.
1,000 mm.	+ 16	— 0
32 in.	+ 0	— $\frac{1}{2}$ in.
800 mm.	+ 15	— 0
64 in.	+ 0	— 1 in.
1,600 mm.	+ 26	— 0
72 in.	+ 0	— $1\frac{1}{8}$ in.
1,800 mm.	+ 29	— 0

(3) Pallets shall all be double-decked and shall permit the entry of forks or fingers of lifting trucks, preferably from any one side, but at least from two opposite sides.

(4) Distance from underside of top deck to ground:—

Small pallets 5 in. (127 mm.) maximum
Large pallets $5\frac{1}{2}$ in. (140 mm.) maximum

(5) The free height for the entry of forks or fingers, from any side, shall be $3\frac{7}{8}$-in. (99 mm.) minimum for all the specified sizes.

(6) The bearing surface of the bottom deck of pallets of all specified sizes shall be equal to or greater than 40 per cent of the overall surface of the top deck.

(7) Minimum openings in the bottom deck of 2-way and 4-way entry pallets, which shall be provided to allow the wheels of a pallet truck to bear upon the ground, are indicated by drawings, examples of which are Figs. 1 and 2 for 32 × 48-in. and 40 and 48-in. pallets respectively.

(8) The width of wings, for lifting, of wing pallets shall be:—

Small pallets	*Large pallets*
$2\frac{1}{2}$ in. (65 mm.)	3 in. (75 mm.)

(9) To facilitate the entry of the wheels of a pallet truck, the members of the bottom deck of a pallet shall be chamfered on each side of the top face in the following way:—

(i) the angle between the chamfered surface and the horizontal shall be 40° ± 5°.

(ii) the height of the vertical face of the member shall be $\frac{1}{2}$ in. ± $\frac{1}{8}$ in. (10 mm. ± 5 mm.).

This recommendation shall apply to a pallet made of any material when the thickness of the member concerned exceeds $\frac{3}{8}$in. (10 mm.).

(10) Non-expendable pallets of the sizes specified shall be capable of supporting the following loads, uniformly distributed.

	Whilst being handled	*When stacked*
Small pallets	1,000 kg.	4,000 kg.
Large pallets	2,000 kg.	8,000 kg.

(11) **Box Pallets**

(i) There shall be two types of box pallets, which may be with or without a lid—viz. (a) those in which the bottom decks comply with the conditions for flat pallets and (b) those with feet. For both types, the overall plan dimensions of the whole pallet shall not exceed
$32\frac{7}{8}$ in. × 48 13/16 in. (835 × 1,240 mm.)
40 15/16 × 48 13/16 in. (1,040 × 1,240 mm.)

(ii) The design of a box pallet shall permit (a) a standard box pallet of the same type and dimensions to be stacked on top of it and (b) a standard flat pallet of the same base dimensions to be stacked on top of it.

Testing of Pallets

For a long period of time there have been considerable differences of opinion on this matter. The discussions at Harrogate were

International Pallet Standardisation—*continued*

successful in that the manner in which the matter is to be dealt was settled in principle and a Working Group was charged with the task of preparing a basic standard specification and testing procedure for I.S.O. standard flat timber pallets. The suggestion is that the specification should stipulate (a) species of timber, (b) finishing quality of pallet material, (c) maximum loading, (d) details of construction, (i) design and (ii) securing of components and (e) branding and marking.

As far as testing procedure is concerned, the present proposal is that pallets complying with the standard specification (when agreed) shall fulfil the requirements of two specific tests viz (a) a deflection test and (b) a "free fall on corner" test.

As stated, one Working Group will attempt to draw up a recommendation on these lines—and it certainly has its hands full! The committee charged a second Working Group with the task of investigating ways and means of testing metal pallets.

Before disbanding, the committee discussed a recommendation by Russia that consideration should be given to the current proposal to establish an international pallet pool. The formation of such a pool has been under consideration in Europe for some time. In the latter half of 1957, the Economic Commission for Europe submitted proposals for establishing a pool through the Committee of European Ministers of Transport to the International Chamber of Commerce and the Chamber in turn referred the matter to a Commission of Transport Users. After two meetings, this body suggested the following guiding principles:

(1) The organisation should be as simple as possible. The international pool might, for example, be controlled by a neutral body, the constitution of which would be studied by the International Chamber of Commerce.

(2) The pool would be open to Users and all modes of transport which used and furnished the necessary number of pallets for the requirements of their enterprises. The pallets of the pool would be exchanged between the different parties, users or carriers.

(3) The pallets should conform to the type chosen (construction and dimension). Their checking and agreement would be entrusted to a responsible body.

(4) The fundamental rule for exchange would be: one empty pallet against a loaded one and vice versa.

(5) Tariffs. In principle, the weight of the pallet would not be charged for and care would be taken to avoid returning empty pallets.

(6) Customs: It should be arranged that pallets pass duty free.

(7) Repairs: Each member of the pool would undertake to (a) put pallets back into use in a good state of repair (checking on receiving and returning of the pallet) and (b) repair, have repaired or replace any pallets damaged in his area.

As stated, this matter was discussed at Harrogate but the decision was reached that the formation of a pallet pool was an enterprise for a body other than I.S.O. to undertake, although steps should be taken to ensure that the pool would consist of I.S.O. standard pallets. It was noted that the Commission of Transport Users had been divided upon whether only one size (40×48 in.) of pallet should be used in the pool, or two sizes (32×48 in. and 40×48 in.). Both, of course, are standard sizes.

Harbour Radar for Port of London

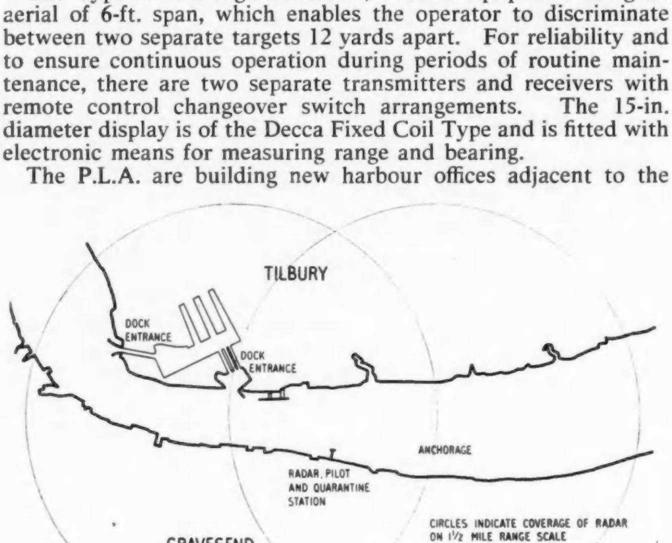
Equipment for Thames Navigation Service

The Port of London Authority have placed a contract with Decca Radar Ltd., for a Decca Harbour Radar Type 33 for the Thames Navigation Service, which will begin operating in May 1959. A full description of this project was published in the April 1958 issue of this Journal. The new radar station, a variation of the type being fitted or already fitted at Liverpool, Hamburg and Southampton, will be one of the most advanced in the world.

The Type 33 is a high definition, X-band equipment using an aerial of 6-ft. span, which enables the operator to discriminate between two separate targets 12 yards apart. For reliability and to ensure continuous operation during periods of routine maintenance, there are two separate transmitters and receivers with remote control changeover switch arrangements. The 15-in. diameter display is of the Decca Fixed Coil Type and is fitted with electronic means for measuring range and bearing.

The P.L.A. are building new harbour offices adjacent to the

The Type 33 Radar Display Unit.



DECCA HARBOUR RADAR AT GRAVESEND

Royal Terrace Pier on the south bank of the Thames at Gravesend. The building is designed round the operations room which will be an integrated port operating centre, collecting, sorting and providing to ships navigational information covering the whole river from the Nore to London Bridge. The scanner of the first radar will be mounted on the roof of the building and the display will provide a detailed picture of the situation in Gravesend Reach. The operations room is designed so that, as a result of experience, further radar stations can be set up above and below Gravesend, feeding additional displays. Eventually, if the demand necessitates, the entire river from the seaward limit of the port to the Royal Docks will be shown on eight radar displays at one central position.

Harbour Radar for Port of London—continued

The choice of Gravesend for the first radar station means that the most critical section of the river will be the first to derive benefit from the operation of these facilities. Through this area where the channel is 1,000-ft. wide, an average of 80 ships pass each way every 24 hours. In addition, there are two entrances to Tilbury Docks, tug stations, ferries, the Tilbury Landing Stage and numerous other harbour activities, while ships also use the reach as an anchorage. It has therefore become essential for the Port Authority to take steps to prevent undue congestion of shipping in this area during periods of low visibility.

A shore radar installed at a well-chosen site can provide a wealth of information and keep the shore authority completely up-to-date with information on the position and movements of all vessels within the area. In co-ordinating the movements of vessels it may sometimes be necessary for the Harbour Master to request a ship to remain alongside or to regulate the time of entry into the port. Once the ship is underway most authorities are agreed that shore assistance should take the form of a steady flow of information passed to the vessel. On board the master and the pilot continue to exercise their control, but are assisted in their decisions by a complete knowledge of the traffic and berthing situation in the area.

Although Harbour Radar is of greatest use when the visibility is reduced to a minimum, it is also of considerable benefit in good visibility. By its use a rapid and comprehensive surveillance of the entire port area can be obtained more readily than by any other method, enabling movements to be co-ordinated and unnecessary delays avoided. The PPI provides a picture which the operating staff knows intimately and where every fixed mark is recognised and moving objects are seen instantly. This information when relayed, helps to supplement the ship's radar or visual information, which may sometimes be confused.

An important capacity in which radar can give valuable guidance is in the assessment of calls for emergency assistance, and the provision of rescue or salvage services, if necessary improvising these at short notice until suitable craft can be brought to the scene. This facility will limit disorganisation of port operations and the movements of vessels not involved in the incident will be unaffected.

Experience of Harbour Radar at a number of important ports throughout the world has demonstrated clearly, both to shipowners and port authorities, that these systems, in conjunction with the recently standardised Marine V.H.F. communication frequencies, offer great advantages and economies both of time and money.

Carriage of Uncrated Cars

Preparing Ships' Holds for Loading Vehicles

For some time, much thought has been given to the problem of stowing in ship unpacked motor cars. The primary needs have been to use freight space economically, to increase speeds of loading and discharge and to ensure the safety of the ship and her cargo.

The increasing export of unpacked motor cars, particularly for Pacific Coast destinations, has been the reason for recent experiments, all designed to augment the carrying capacity of export vessels. Early this year, the s.s. "Lake Pennask," a 9,950-ton vessel, was converted for this purpose by the construction of additional wooden decks in each of her five holds. Under conventional loading conditions, the ship would have carried perhaps 250-300 cars; converted, she was able to carry nearly 900.

The latest device, where temporary and intermittent use of general cargo vessels is required, is the erection of false decks using tubular steel scaffolding to which wooden decking is clamped (Figs. 1 and 2). The first vessel in which the experiment was made was the s.s. "Cape Hawke"; in which three additional decks were constructed in each of Nos. 2, 2a and 4 holds. This method has been developed by Tatham Bromage and Co. Ltd. jointly with Acrow (Engineers) Ltd. Other types of false deck erection do, of course, provide similar facilities but one of the advantages of this latest method is that the decks are quickly and easily erected and dismantled and these operations do not necessarily require skilled labour.

For an average type of cargo vessel of between 10,000 and 12,000 tons d.w., the scaffolding required weighs about 100 tons and the timber a further 100 tons, providing stowage for between 600 and 700 cars. In general, provided no unexpected difficulties occur, installation takes about four days.

Construction is flexible and adjustable so that the decks can be moved from one part of the ship to another if required, or even from one ship to another. It has the minimum of loose fittings and when dismantled can be packed down into a small space. The weight of this new structure is only about half that of the all-

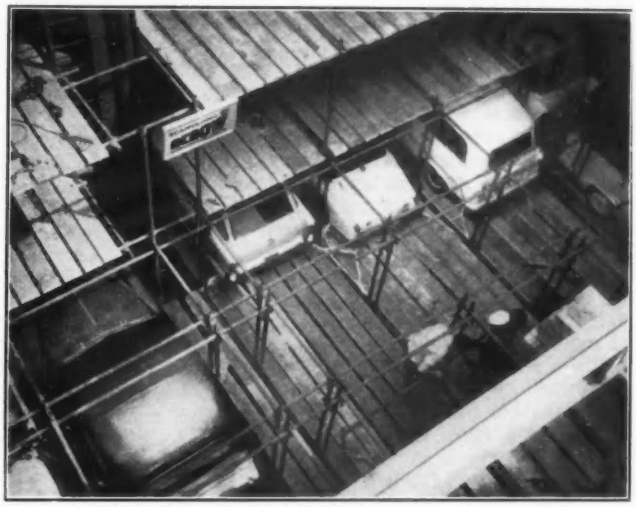


Fig. 1.

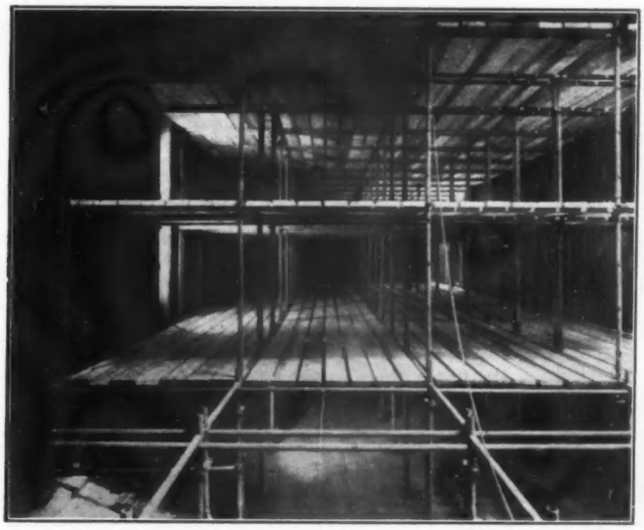


Fig. 2.

timber structure previously employed, with the result that three or four tiers can be stowed in the lower holds. The boards used for decking have spaces between, permitting light to filter through, and it is a simple matter for ship's personnel to pass between the cars during voyages, to carry out inspections and check lashings.

Viewed from the angle of safe carriage—and our export trade relies on goods being delivered in sound marketable condition—this method offers many advantages. The horizontal tubular members of the structure provide so many points to which cars can be lashed that movement on passage should be virtually eliminated. It is also claimed on behalf of the new method that speeds of loading and discharging are considerably increased. Finally, the cost is less, one reason being that none of the material used is expendable.

Ore Handling at the Hartlepools

New Methods Reduce Operating Costs

Since the construction of a new deep water berth two years ago, iron ore imports handled at the Hartlepools, have increased from 130,000 tons to 600,000 tons per annum and are expected eventually to rise to 1,000,000 tons. This increase has been made possible to a large degree by the mechanisation of the grab-feeding operation for clearing ships' holds.

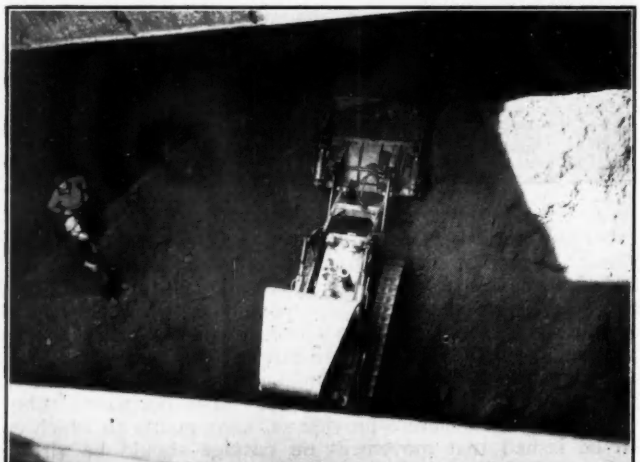
Using earth moving equipment on crawler tractors, with only minor modifications, the turn-round time of 7,000 to 11,000 ton cargo vessels has been reduced by as much as 16 to 24 hours. Demurrage on ships of this size can cost as much as £700 per day, so that the Charter Parties benefit by this saving, and also by having vessels cleared for further cargo more quickly.

The predominant type of vessel at present carrying iron ore to the Hartlepools is of the tramp steamer class with the usual 'tween decks, tunnel holds, long ends and deep wings, all of which contribute to slowing down the final stages of discharge. Until two years ago, the usual method of discharging iron ore was to grab as much of the cargo as possible, then engage tubbing gangs to shovel out the ore from stowage areas inaccessible to the grabs.

To speed up the turn-round of ore vessels, the British Transport Commission decided to experiment with mechanical aids, trying calf-dozers, bulldozers, overloaders and other earth moving and materials handling equipment. After a few months of employing hired plant, a small fleet of machines was built up for use in the ore vessels, with very encouraging results. In the light of these experiments it was decided that tracked machines were essential as, owing to the density of the cargo, wheel tractors were too prone to wheel spin when using maximum power to push or dig into the ore.

The first machine purchased was an Anthony Loader Dozer, with International BTD-6 crawler tractor, a dual purpose machine which can be used with either a blade or over-loading bucket. As a dozer it is useful for pushing ore from the long ends to the square of the hold and, where two holds are not divided by a bulkhead, for pushing the ore from the smaller to the larger of the two holds. As an overhead loader, it is extremely effective in tunnel holds for throwing the ore out from the wings and long ends. The first machine was sufficiently satisfactory for a second machine of the same type to be purchased four months later.

A little over a year ago, a demonstration was given on board ship with an International Drott Skid Shovel equipped with 4-in-1 bucket. As the name implies, this machine is designed to perform four distinct operations (1) dozer (2) digging and loading shovel (3) clam grab (4) carry-type scraper—in addition, when in the grab position, it can be used to back-drag or doze in reverse. Following the demonstration 2 of these machines were added to the fleet.



Back-dragging ore to the square of the hold.

The outstanding features of the Drott machine, when considered for iron ore handling, are the back-dragging and dozing operations, the other positions being used seldom. Before putting a Drott into the hold of the ship, the usual practice is to grab out the ore from the square of the hold to a depth of seven or eight feet. The machine is then put in to drag down level into the square, the ore left heaped in the wings and ends. This procedure can be followed through to the completion of discharge, even to the extent of cleaning up at ceiling level.

Iron ore is possibly the toughest commodity on which any machine can be used, a vastly different proposition from the normal excavation work for which the machines were originally designed. Nevertheless, few modifications have had to be made. It was necessary to provide a $\frac{1}{4}$-in. steel plate canopy for the operator but it was found inadvisable to do so for the engine. Ordinary high grouser tracks were found to cause excessive damage to tank top sheathing and ceiling timbers and low profile grouser shoes are essential on all machines.

Dust content of the iron ore must always be an important factor, as the highly abrasive dust tends to enter into bearings and to penetrate fuel and hydraulic systems, and the authorities have found it a valuable precaution to institute 1,000 working-hour general inspections and overhauls.

The additional cost of modifications and overhauls have been far outweighed by the economies in time and labour. Although manning scales are maintained, less labour is required than hitherto for the discharge of an ore vessel and even fewer actual man hours are involved for any one vessel. (Grabbing gangs are smaller than tubbage gangs).

Reports indicate that these machines and similar methods are being tried at certain Continental ports, but the Hartlepools is believed to have achieved the most complete efficiency in this direction so far. As a consequence, iron ore is becoming one of the most important bulk commodities being handled at these docks.

Manufacturers' Announcements

Hydroconic Tug for Australia

An order for a twin screw all welded steel tug of Hydroconic design has been placed with Messrs. Seawork Limited by J. Fenwick & Co. Pty. Limited, Sydney, New South Wales. It will be built at the Gateshead-on-Tyne shipyard of T. Mitchison Limited.

The tug, which is intended for coastal and harbour duties in Australia, will be powered by two marine diesel engines each developing 552 b.h.p. at 560 rev./min. built by Mirrlees, Bickerton & Day Limited of Stockport. The drive to the propeller shaft will be through the medium of a flexible coupling and oil operated reverse reduction gearbox to give 525 s.h.p. at the propeller at a speed of 185 rev./min., and will be arranged for remote control with quick changeover arrangements for engine room control in an emergency.

The windlass and capstan will be hydraulically operated and the steering gear will be of the electric hydraulic type. A spring towing hook is to be fitted and life saving apparatus will include a 16-ft. aluminium lifeboat. The principal dimensions are length B.P. 88-ft., breadth moulded 24-ft. 9-in., depth moulded 12-ft. 6-in. and draught aft 12-ft. 6-in. The vessel will be built to Lloyd's Class 100 A1 (for towage services) and will also comply with the latest Ministry of Transport and Australian Regulations.

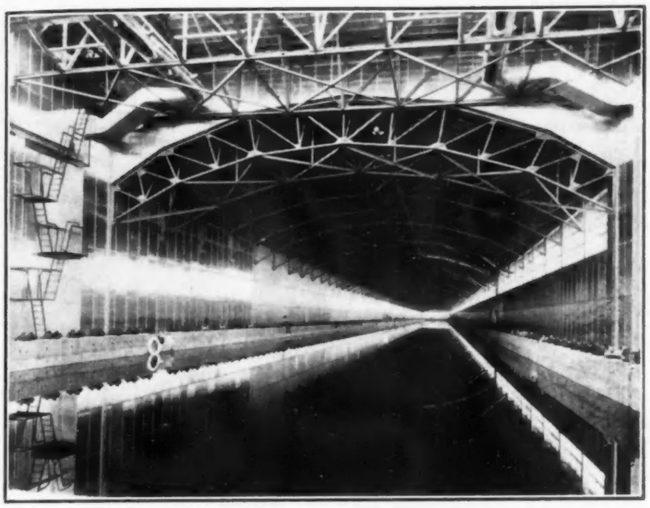
An interesting point is that this is one of the first orders for a twin screw hydroconic tug with twin rudders for Australia, where hitherto single screw tugs have predominated. It may prove to be the beginning of a general trend towards twin screw tugs on the grounds of their high degree of manoeuvrability in restricted water, especially when handling the larger type of vessel.

Manufacturers' Announcements—continued

Aluminium Roof for Ship Testing Tank

The roof structure of the Ship Testing Building at Feltham is believed to be one of the longest built in aluminium. It consists of 98 curved roof trusses with a span of 67-ft. and covers an area 1,460-ft. long by 67-ft. wide. The fabrication of this structure was undertaken by Head Wrightson Aluminium Ltd., a subsidiary of Head Wrightson & Co. Ltd., for the Ministry of Works.

The temperature in the Testing Tank Building is kept at a constant figure of 70°F. and the highly humid atmosphere resulting in heavy condensation in the roof members presented corrosion and maintenance problems, and these considerations



View of ship testing tank at Feltham.

coupled with loading restrictions dictated the use of aluminium as the most suitable material.

The roof is supported by a steel structure supplied by Head Wrightson Teesdale Ltd. and the building is a good example of a composite construction in which approximately 105 tons of aluminium was used. The aluminium trusses and lattice girders were fabricated in very large sections so that work on the site and erection time could be cut to a minimum. Mild steel site bolts were heavily sheradised and over 86,000 aluminium rivets were driven during the construction of the roof members.

The plates and sections were supplied by the British Aluminium Co. Ltd., I.C.I., Metal Division and T.I. Aluminium Ltd.

Dunnage Air Bags

A new form of dunnage enabling goods in transit to ride literally on air is now being made in Britain. It is an inflatable rubber bag which can be used to prevent the movement of goods in railway wagons, lorries and ships. The R.F.D. Company Limited of Godalming, Surrey, are the manufacturers.

Freight is normally blocked or braced in a variety of ways the most common being the use of wooden shoring. This type of dunnage is a slow and expensive operation. The shoring too often collapses when subject to high speed vibration or shunting impacts.

The new airbags are slightly inflated to shape, slid into position and then blown up to the required pressure according to type of cargo. An air supply must be available for inflation but no skill or special tools are needed.

It is claimed that, not only is a fraction of the time previously required now necessary to shore up goods, but the bags also absorb shocks and vibration, eliminating cargo damage and the loss of time, money and customer goodwill involved. The bags are deflated in seconds by the unscrewing of a valve and can be folded in a small parcel to be returned for further use.

The bag consists of an inner bladder and a tough outer casing which affords protection against abrasion, oils and petrol, weathering, etc. It can be made in various dimensions from 2-ft. x 4-ft. to 4-ft. x 8-ft. When inflated, single bags can be used to brace gaps up to 16-in., but for larger gaps superimposed bags are necessary. Total weight is approximately 25 lbs.

Barge Train for West Africa

An integrated barge train, built by Yarrow & Co., Ltd., to the order of the United Africa Co., Ltd., London, recently completed trials in Loch Long. The train, which comprises the pusher craft "Gondola" and eight barges, measures 630-ft. overall and has a beam of 66-ft. It will be based at Burutu, the river port of the United Africa Co., Nigeria, and will operate on the Niger and Benue rivers.

A comprehensive series of model tests were carried out at the National Physical Laboratory to determine the optimum performance of the craft which was designed by the late Mr. Alexander Kari. The company were the first river operators to introduce "push towing" to Nigeria in 1950, developing this system from small trains of 60 tons up to 2,800 tons.

The "Gondola" is a twin-screw tunnel-type vessel specially designed for shallow-water conditions, and has an overall length of 105½-ft., a moulded breadth of 23-ft., depth moulded to the main deck of 7-ft. and an extreme draft of 4½-ft.

The propelling machinery consists of two normally aspirated uni-directional Ruston-Paxman type 16 RPHM engines, each driving a propeller shaft through an oil-operated modern wheel drive combined reverse-reduction gearbox, with a flexible coupling fitted between the engine and gearbox. Each engine has a continuous output at the gearbox coupling of 560 b.h.p. at 1,250 r.p.m. and is provided with a closed system of fresh water cooling and independent heat exchangers.

To ensure maximum manoeuvrability of the barge train, the "Gondola" has two steering and four flanking rudders, all of partially balanced double plate type, actuated by hydraulic steering gear supplied by Vickers-Armstrongs (Engineers) Ltd. The two steering rudders can be operated either simultaneously or independently and the four flanking rudders are coupled for operation in pairs. Incorporated in the "Gondola" is radio telephone equipment of Redifron type for communication with the company's headquarters at Burutu and their principal stations in the service area.

The operational formation of the train comprises nine units, consisting of two leading barges, four intermediate box-barges, two trailing barges and the pusher craft. The barge units are each approximately 130-ft. long, have a breadth of 33-ft., a depth of 8-ft. and a draft of 5-ft. Each barge has three transverse watertight bulkheads forming two cargo holds and the forward and after peak compartments. No longitudinal bulkheads are fitted. The cargo hatches each 36-ft. by 18-ft. are of roller-beam type, with 2-in. thick wood covers. A two-berth cabin and galley are arranged on each box and trailing barge.

Four Ransome & Rapier cranes are provided for handling cargo, each capable of lifting 1½ tons at 33-ft. outreach. One crane is fitted on each of the four barge units forming one side of the train, positioned between the cargo hatches. When loaded to a draft of 5-ft. the complete train has a displacement of approximately 4,650 tons and will be capable of transporting a total of 3,660 tons of cargo. For the purpose of the trials held at Loch Long, the eight barge units were suitably sub-divided by temporary wash-bulkheads and ballasted with water.

It is recognised that "push towing" offers advantages over the more conventional methods of handling dumb barges, the pusher tug having more control over the tow, which improves manoeuvrability and reduces the risk of grounding in narrow and tortuous channels, particularly during the low water season in tropical areas. An integrated "push tow" train also offers less resistance, so that a higher speed is obtained for the same power output or, alternatively, the same speed can be obtained with a greater payload.

Manufacturers' Announcements—continued

Submarine Power Cable for Canada

In the December, 1956, issue of this Journal, a description was published of the design and installation of the submarine cable used for supplying bulk electricity to Vancouver Island from the mainland of British Columbia. A second circuit in the 138,000-volt power link was put in commission recently. The cable was manufactured for the British Columbia Electric Company by British Insulated Callender's (Submarine Cables) Limited at their Trafford Park works, Manchester. The H.M.T.S. "Monarch" was chartered to transport the cable from Manchester to Vancouver and to lay the four continuous lengths between the mainland and Vancouver Island.

The existing submarine power cables, which were also manufactured and installed by the BICC Group, have been in operation since September, 1956. These, with the two continuous lengths of 16 miles and two of about three miles, required to complete the second circuit, with an additional spare cable, bring the installed length of cable for this project to a total of over 133 miles. The combined current-carrying capacity of the two circuits, is now 250,000 kilowatts, a capacity greatly in excess of that of any similar undertaking in the world.

Mobile Cranes for Liverpool

Ransomes and Rapier Limited, Ipswich, have recently received an order for six RapieR 4 Standard Mobile Cranes from the Cunard Steam-Ship Company Limited, Liverpool. These cranes are required for shifting cargo to and from ship's side to dock sheds and for loading and unloading railway wagons and motor lorries. The cranes will be mounted on pneumatic tyres and give exceptional manoeuvrability through a wide angle steering lock which enables them to turn so sharply that in effect they slew on their own road wheels. The drive is diesel-electric with separate motors for each motion enabling loads to be handled with speed and precision.

Harbour Craft for Overseas Service

Aluminium Alloy Launch for Jamaica

During the past two years Universal Shipyards (Solent) Ltd., have designed a number of launches based on their standard light alloy hulls. The latest craft has been built for pilotage duties at Port Esquivel Terminals, Jamaica, and also for checking by echo-sounding the progress of dredging work at the port.

The vessel, which has been named "Katharine," has an overall length of 35-ft., a beam of 10-ft. 8-in., and a draught of 2-ft. 9-in. Constructed of light alloy, supplied by the Northern Aluminium Co., Ltd., Banbury, her decks are of positive grip pattern to give a safe non-skid surface. The accommodation has been divided into two cabins. The forward cabin includes the helmsman's position and accommodates the operator of the echo-sounding

The "Katharine" undergoing trials.

equipment, while the after cabin has seating for eight, or alternatively can provide two bearths when necessary.

Power is supplied by a Single Perkins diesel motor developing 100 h.p. at 2,000 r.p.m.; this is equipped with a heat exchanger unit to provide closed circuit fresh water cooling. The motor is installed centrally between the two cabins, and the engine room is lined with glass fibre sound insulating material. The launch, which has a service speed of 13/14 knots, carries 150 gallons of fuel in two separate tanks. A heavy duty solid rubber fender is supplied by Messrs. Firestone and polyphor expanded material is fitted under the side decks to provide positive buoyancy.

Launch Tug for the Sudan

Trials were recently carried out on the River Thames of a steel launch tug built by the Thames Steam Tug and Lighterage Co., Ltd., at Brentford, Middlesex, under sub-contract to Thames Launch Works. The vessel has been named "Haris" and will soon take up service in the Sudan.

The launch-tug "Haris."

With an overall length of 32-ft., beam of 10-ft. and moulded depth 4-ft. 9-in., the fully loaded displacement is 10 tons. The hull is of mild steel construction throughout, designed on the single chine principle with a good sheer forward, raked stem and rounded stern. Accommodation comprises fore peak, engine room, control cockpit with a permanent wooden awning, a store and after peak.

The launch is propelled by a Thornycroft 6 cylinder marine diesel engine derated to 65 s.h.p. at 1,300 r.p.m. to suit local tropical conditions and fitted with a 2/1 reduction gear. A closed fresh water cooling system is employed with keel cooler and the two fuel tanks have a total capacity of 150 gallons.

During trials a mean speed of 8 knots at full power was attained and a bollard pull of 15 cwts. was registered when running the engine at 1,000 r.p.m. on a developed power of 49 s.h.p. The vessel was designed by Courtney, Hughes & Partners Ltd., and is the first export order to be completed by the builders since they embarked upon producing powered craft for the home and overseas market.

Salvage and Mooring Vessel for Kuwait

A small specialised craft was recently launched at Messrs. Whites Shipyard (Southampton) Ltd., for harbour service at Kuwait in the Persian Gulf. The vessel will be used for salvage work at the port and also for recovering the heavy moorings and buoys required for the large tankers.

Fitted with twin 35 h.p. "Harbormaster" propulsion units, the vessel has an overall length of 90-ft., beam 30-ft. and depth 8-ft. An 18-ton Coles diesel electric crane with a 40-ft. jib will be mounted on the fore deck, MacTaggart Scot 10 ton hydraulic capstans on the after deck, and the vessel will be equipped with four anchors each controlled by hydraulic winches. Fitted aft is a 180 ton per hour fire-fighting and salvage pump supplied by Merryweather & Sons, Ltd., driven by a Rootes-Lister diesel engine. Construction is of steel throughout and the vessel has been built to Lloyds' Classification and supervision.